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13. ABSTRACT *(Maximum 200 words)*

This document includes material assembled as a completion report of the Corps of Engineers Structural Engineering Conference (CESEC 91) held in St. Johns County, FL, 8-12 July 1992. Under this cover are the agenda of the conference, abstracts and papers presented, a description of the exhibits, posters and demonstrations displayed, a list of attendees, and an evaluation of the conference by the participants.

The 5-day conference included the presentation of 112 papers, 32 visual displays, six training sessions and two leadership forums on the state-of-the-practice in structural engineering within the Corps of Engineers and other agencies in attendance. The presentations were given as formal talks or informally through visual displays and demonstrations. A challenge workshop was conducted one afternoon where a select group of structural engineers were presented with two emergency situations which could likely occur. These teams of engineers were asked to provide an immediate assessment of the situations and both a short-term and long-term solution to the problems. Leadership forums were conducted for chiefs of structural design from each office for both military and civil works areas.

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The conference was attended by 263 engineers representing 12 division and 36 district offices, HQUSACE, the Construction Engineering Research Laboratory, the Cold Regions Research Laboratory, the Waterways Experiment Station, and non-Corps offices including the Federal Energy Regulatory Commission, Soil Conservation Service, US Bureau of Reclamation, Civil Engineering Research Foundation, Washington University, Lehigh University, Ohio State University, University of Colorado, Intergraph Corp., Black and Veatch, Goldberg & Simpson, and Lester B. Knight & Assoc.

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Responsibility for Design of Steel Structures

by
David B. Ratterman[1]

(This paper was received just prior to publication and is included as Appendix E.)

[1] Goldberg & Simpson.

High-Strength Bolted Connections Clarifying Issues

by

Ray Decker, PE[1]

Abstract

The purpose of this report is to share with Corps designers supplemental guidance for the design, detailing, and installation of high-strength bolted connections (HSBC) and to help clarify confusion that has been caused, in part, by significant changes in terminology, design philosophies, and installation recommendations contained in the Research Council on Structural Connections (RCSC) Specification for Structural Joints Using ASTM A325 or A490 Bolts (American Institute of Steel Construction 1988). In addition, confusion has resulted from the American Institute of Steel Construction (AISC) having two different manuals of steel construction. One manual uses the Allowable Stress Design (ASD) method (AISC 1989), and the other uses the Load and Resistance Factor Design (LRFD) method (AISC 1986). This report will begin with a brief history of HSBC. The current criteria for bolts loaded in shear, tension, and in combinations will be reviewed and some of the design, detailing, and installation requirements for slip-critical (S-C) and bearing/shear (B/S) joints will be given. Some special conditions of HSBC joints will be given. Finally, recommendations concerning the selection of joint type and for high-strength bolting in general will be given. The recommendations will include identifying when S-C joints should be used and will give the benefits of using snug-tight B/S connections.

History of HSBC

Rivets were the principal fasteners used in the early days of structural steel construction. It was known that the cooling of hot-driven rivets produced a clamping force. In 1934 Batho and Bateman were the first to suggest that high-strength bolts could be tightened enough to also provide a clamping force that would prevent slip in the joints (Kulak, Fisher, and Struik 1987). Little was done with HSBC, however, until 1947 when the Research Council on Riveted and Bolted Joints (RCRBSJ) was formed. The first RCRBSJ report, approved in 1951, permitted a like number of prestressed American Society for Testing and Materials (ASTM) A325 bolts in lieu of hot driven ASTM A141 rivets. This type of prestressed joint where member forces are resisted by the friction between the faying surfaces was termed a "friction" connection. It became very popular. The method used to determine tension in friction bolted connections was a torque-to-tension relationship that was given in a table in the 1951 specifications termed the "calibrated wrench." The table was quickly withdrawn in 1954 because the torque-to-tension ratio had been found to vary by as much as 40 percent. This method was entirely deleted in 1980 because of the complexity of

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calibration and inadequate inspection procedures. Also in the 1954 specification, it was recognized that resistance to slippage is necessary only in special conditions - thus the use of "bearing" connections was introduced. In the 1964 specification, the higher strength but less ductile ASTM A490 bolts were introduced to recognize the need for connecting high-strength steel without resorting to very large connections.

Recent HSBC Criteria

Effective with the 1985 RCSC specification, "friction" and "bearing" connections were changed to S-C and B/S connections. This was done in an effort to focus attention upon the real manner of performance of HSBC loaded in shear. The first RCSC specification that utilized a strength design approach, termed LRFD, was published by AISC (1986) in the LRFD manual. The 1986 RCSC spec utilized the 1985 RCSC (ASD) spec - changing only the design portion of the specification. The LRFD design for slip resistance is the same as the ASD, however, significant changes were made to the requirements for the design of B/S connections. The current 1988 RCSC LRFD spec has been adopted by AISC.

HSBC Design

Design philosophy

The philosophy of design for HSBC is to begin with checking the joint strength in shear and bearing. Then, for S-C connections, the resistance to slip is checked. This is true for both the ASD and LRFD methods. The reason for this procedure is that high clamping forces with high coefficients of friction might create slip resistance that exceeds the shear strengths of the fasteners or bearing strength of the connected material. This is conservative, mathematically the slip resistance may exceed the shear or bearing capacity, but in reality the fasteners would not be subject to shear and bearing prior to slip, and the combined effect of frictional resistance with shear and bearing is not considered.

Comparing S-C & B/S LRFD joints

Generally, faying surface conditions for S-C are classified as Class A and Class B. These classes are defined in the RCSC. Generally, when an S-C Class A coating is used, slip controls the number of bolts needed in a connection, and, if an S-C Class B coating is used, shear and bearing controls.

Connections Subject to Shear

S-C connections

General. The S-C name accurately reflects that this type of joint is used only when resistance to slip is critical. To prevent slippage, S-C bolts are tensioned to 70 percent of the strength of the bolt providing a clamping force to the joined elements. Slippage is resisted by friction on the faying surfaces of the elements, the amount depending on the bolt tension force and the coefficient of friction of the surfaces.

Behavior. The load-elongation curve of an S-C joint is shown in Figure 1.

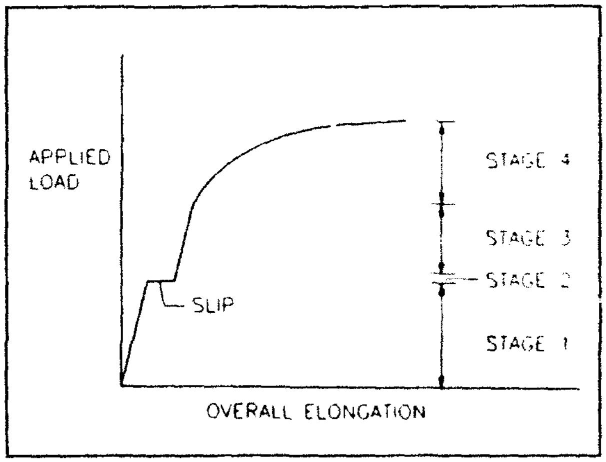

Figure 1. S-C connection behavior

This curve is divided into four segments corresponding to the four stages of loading. In stage one, friction prevents slip so the bolts are not subjected to shear nor the connected material to bearing. In stage two, the applied

force overcomes the developed friction and the bolts make contact with the plate surfaces. Stage three represents elastic deformation of the fasteners and the plates. In stage four, a B/S-type failure occurs either by plate fracture or shearing of the fasteners. At failure the bolt preload has been released by shear deformation and plate yielding; thus the initial preload has no significant effect on the final failure load. Although S-C joints have considerable strength beyond the slippage stage (stage two), that stage should be considered failure for connections that cannot tolerate slippage.

B/S connections

General. B/S connections rely on bearing of the bolt shank on the surfaces of the connected elements. B/S connections should be used when slip resistance is not a performance consideration. B/S connections may be tightened "snug-tight" but can be "fully pretensioned" if desired by the designer. Fully pretensioned B/S connections are not required to meet the faying surface conditions of S-C connections nor are they required to be tested.

Behavior. Figure 2 shows a typical load-deformation diagram for a snug-tight B/S connection.

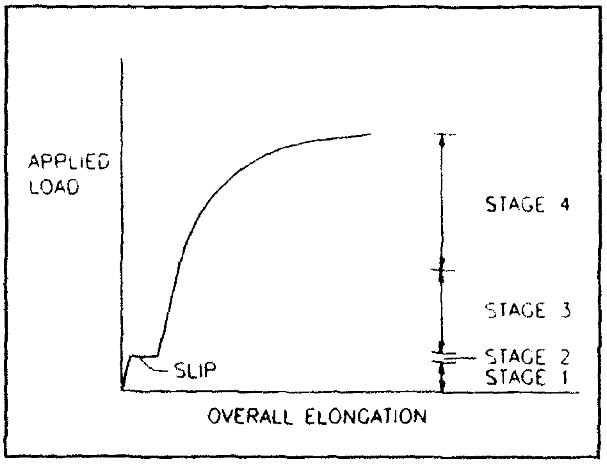

Figure 2. B/S connection behavior

The B/S curve is divided into the same four loading stages as the S-C curve, however, with the lower tension force, slip will occur much earlier in the B/S connection. This is not detrimental since slip can be tolerated.

Connections Subjected to Direct Tension

General

High-strength bolts loaded in direct tension *must* be pretensioned. The direct tension may be a static or a repeated loading. The bolts may also be subjected to shear, in which case the combined effects must be considered. Under certain conditions, the tension loading may cause prying action. If so, the prying action must be considered during design.

Behavior

Full pretensioning of a bolt results in an axial force or preload in the bolt. This load exists before the application of external load so that the bolt is considered prestressed. Prestressing creates contact pressures between the plates being joined. When external tension loads are applied, this contact pressure is decreased, however, very little increase in bolt tension occurs until the external load exceeds the internal pretension load. This process is illustrated in Figure 3. Above the pretension load level the load in the bolt equals the external load.

Connections Subjected to Combined Effects

Direct tension and shear

In B/S type connections, the strength limits of fasteners subjected to shear and tension are determined by equations which give elliptical interaction curves. The equations are in terms of the shear stress and are solved for the tension strength limit.

Direct tension and prying action

In a great many direct tension connections prying action occurs when the contact sur-

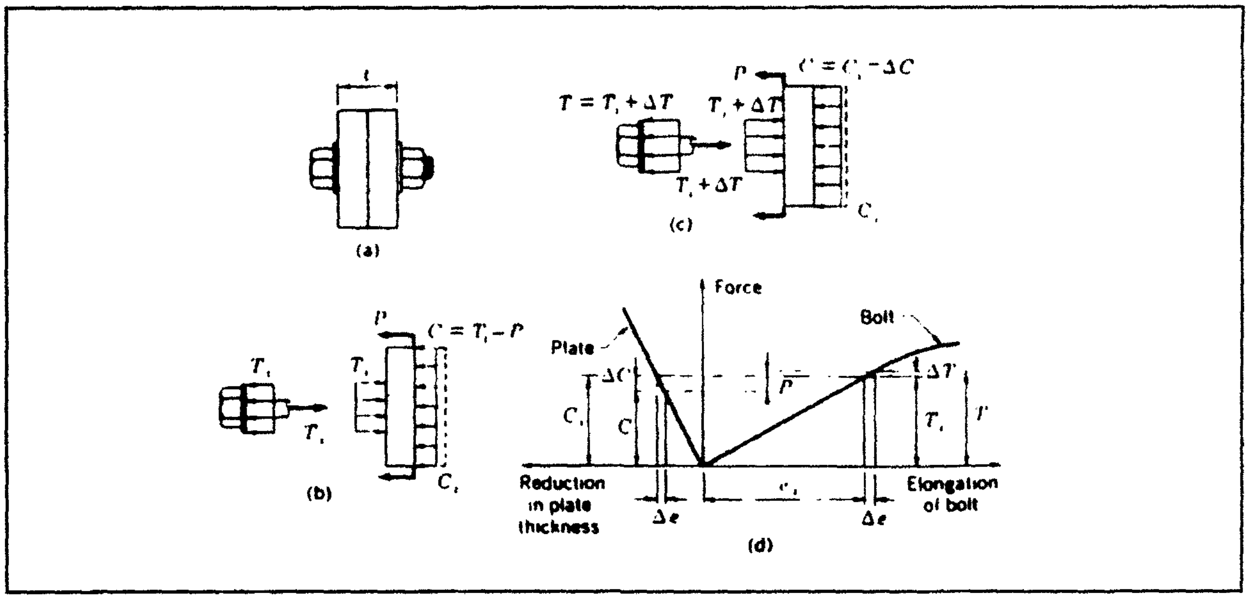

Figure 3. Direct tension connection behavior

faces begin to separate. A connection where prying action may occur is shown in Figure 4.

When an external tensile load applied parallel to the stub tee web reduces the contact pressure between the stub tee flange and the beam flange in an unsymmetrical way, prying action may be developed. The AISC manuals' provisions increase the axial force in the bolts with prying action but do not account for the distortion of the connected parts which causes bending in the bolt and in the bolt head and nut. This bending can be appreciable even if the bolt force is not increased appreciably. For this reason, using high strength bolts to resist external tension should be avoided whenever possible.

Direct tension and repeated loading (fatigue)

This combination of loadings where bolts loaded in tension are subject to fatigue should be avoided. The undesirability of this load combination is reflected in tension strength reductions of 50 to 65 percent in the 1988 RCSC provisions.

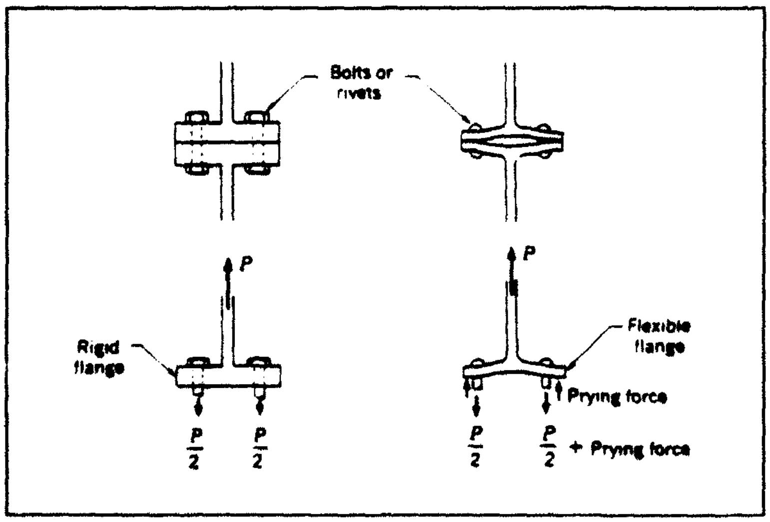

Figure 4. Prying action on fasteners

Special Conditions

Filler plate dilemma

The criteria concerning filler plates can be interpreted as incompatible with the current design philosophy for HSBC. The current RCSC specifications state that "The design of HSBC under this specification begins with consideration of strength required to prevent premature failure by shear of the connectors or bearing

failure of the connected material. Next, for connections which are defined as S-C, resistance to slip load is checked." Current design philosophy is to check connections as B/S, since this is the final mode of failure, even in S-C connections. The possible incompatibility arises when the AISC specifications state that "When bolts or rivets carrying loads that pass through fillers thicker than 1/4 in., except in connections designed as S-C, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough bolts or rivets to distribute the total stress in the member and the filler, or an equivalent number of fasteners shall be included in the connection." This criteria could be interpreted to mean that if shear connections with fillers are designed as S-C connections, development of the plate is not required. The criteria should be interpreted to mean that all shear connections must be designed as B/S, and slip resistance must be checked if it is a requirement. If effect, there is no exception for connections with fillers.

Bolts in combination with welds

Welds will not share loads equally with bolts in B/S connections. To achieve bearing of the bolt against the material being joined or shear in the bolt, the material being joined must slip. Slip cannot occur until the weld fails. S-C bolted connections may be used in combination with welds. Weld shear deformation capacity and the observed values of slip in bolted joints are about the same. Weld shear failure can be expected to occur at the same time the bolts slip into bearing. If the connection was designed to be slip resistant, then this would constitute failure. The 1986 RCSC LRFD specifications did not provide a method of combining bolts and welds. S-C connections were designed for serviceability while the welds were designed for ultimate strength. The 1988 RCSC LRFD specification allows designers to design S-C connections in combination with welds if the design for slip resistance is done using factored loads. The designer must be aware of the fact that if bolts and welds are used in combination, they should be designed using the same factor of safety.

RCSC Selection

Selection of joint type

S-C joints should be used only when resistance to slip is deemed by the designer to be critical to the serviceability of the structure.

- Applications where S-C connections may be desirable are:
  - **Joints subject to fatigue.** In most applications, wind resisting connections are not subject to the number of high-stress cycles that would cause fatigue. On the other hand, fatigue needs to be considered for highway bridges.
  - **Joints where welds and bolts share load.**
  - **Joints where slip causes intolerable misalignments.**
  - **Joints with oversize and slotted holes.** All connections with oversize holes and slotted hole loaded parallel to the slot axis must be designed as S-C.
- The use of the B/S connections, whenever possible, is highly encouraged. Potential benefits over S-C connections are:
  - **Fewer fasteners generally are required.**
  - **Less installation labor and equipment are required.**
  - **Only visual inspection is needed.**
  - **Inspection can occur after installation.**
  - **Retightening of bolts is much less likely.** This can result in a shorter installation period.
  - **There are cases where connection slip is desirable.** Beam to column joints of nonmoment resisting frames are an example.
  - **There are cases where minor slippage does no harm.** The serviceability of the structure is not affected. With standard holes the probable slip

is much less than 1/16 in. which is highly tolerable for most structural systems.

Identification Required on Contract Drawings

Bolts in S-C joints or bolts subject to axial tension should be clearly identified on the contract drawings. B/S connections which are to be fully tensioned should also be clearly identified on the drawings. It should be noted that AISC does not have a standard method for identifying S-C joints with the threads excluded from the shear plane. The designer must identify this type of connection by providing notes that clearly identify the connection type.

HSBC Installation

Installation of B/S connections

The two types of B/S connections are snug-tightened and pretensioned. The following is a discussion of each.

- **Snug-tight.** Bolts in all HSBC joints must be brought to the snug-tight condition. For B/S connections, snug-tight can be the final installation requirement. This condition is defined as the tightness that exists when all plies in a joint are in firm contact. This may be attained by a few impacts of an impact wrench or a man's full effort using a spud wrench. The snug-tight condition is obtained by installing bolts in all holes and bringing them to a snug condition before pretensioning is applied. Snug-tightening shall progress systematically from the most rigid part of the connection to the free edges. Retightening shall follow the same sequence until all bolts are simultaneously snug-tight.

- **Pretensioned.** If the designer so designates, B/S joints may be fully pretensioned. When pretensioning is designated, bolts will be installed in accordance with the RCSC specifications requirements for S-C connections as follows. However, the requirements for inspection of faying surface conditions and testing to determine the actual level of pretension is not required.

Installation of S-C and direct tension connections

S-C connections and connections subjected to direct tension must be tightened to 70 percent of their minimum required tensile strength. A table of the tension force required for various bolt sizes and grades is available in the RCSC specifications. The requirements for faying surface conditions for S-C connections is not required for direct tension connections, but direct tension connections are required to be inspection tested to verify pretension.

There are three common methods of bolt installation. All three methods require field calibration to verify bolt pretension. The calibration is done using a direct bolt tension measuring device such as the Skidmore-Wilhelm Torque-Tension Tester. Demonstration tests, as outlined in the RCSC specifications, must be performed to verify that the tensioning procedure provides at least 105 percent of the required tension force. In all methods, the same procedure shall be used to bring connections to the snug-tight position. The three methods are:

- **Calibrated wrench method.** Calibrated wrench was reinstated as a pretensioning method in the 1985 RCSC specification. However, more detailed requirements were included than when this method was previously used. A wrench is calibrated, on a daily basis, to provide the torque necessary for a tension in the bolt equal to 105 percent of the required tension before it stalls. The wrench must be recalibrated when significant difference is noted in the surface conditions of the bolts threads, nuts, or washers. This method is highly sensitive to the surface condition of the turned part and of the gripped material, hence a hardened washer must be placed under the element turned in tightening. Even with the provision of

washers, the tension has been found to be quite variable for the same torque. For this reason, the RCSC does not recognize standard torque-to-tension tables.

- **Turn-of-nut method.** In this method we begin at the snug-tight position and turn the nut a predetermined number of turns to achieve the required tension. When the turn-of-nut method is used, the demonstration testing must show that the method of estimating the snug-tight condition and the turns required from snug-tight develop a tension in the bolt of 105 percent of the required tension. There are two main problems with this method. First, snug-tightening is not precisely defined and can produce a highly variable initial clamping force depending on surface condition and method of nut turning. In fact, small-diameter bolts (1/2 and 5/8 in.) can be overstressed by tightening to the snug-tight position. A different installation procedure, such as using a small torque wrench, could be used to avoid bolt damage. The second problem with the turn-of-nut method is that the number of turns is a function of many variables such as accuracy of snug-tightening, bolt grade, and bolt length. Although this method cannot be relied upon to give uniform tension, it will result in significant pretension in all bolts and is, in that sense, reliable. It is preferred by some experts even though they agree that bolt overstress frequently results.

- **Direct tension indicator (DTI) method.** This is a relatively new method. It has received RCSC acceptance in the 1985 specification and is covered by ASTM Standard F959. A special washer is installed between the element to be turned (nut or bolt head) and the clamped metal. The washer has protrusions of a given height in the unstressed position. As the bolt is tightened, the protrusions are compressed and narrow the gap. The gap width is calibrated to the tension in the bolt - thus the bolt is tightened until the specified gap (and tension) is achieved. Often, experienced installers can, by visual inspection, arrive at the required gap without need for measurement. This method appears to be the most accurate of all three. Possible disadvantages are the added cost of the DTI washer, the possibility that the washers are altered in the field before they are used by hammering or filing the protrusions. Also, since the protrusions usually go into plastic deformation at the required bolt pretension, DTI washers are useless after one load application. This makes loss of tension difficult to detect and verification of a retension force impossible.

Alternate fasteners

The RCSC specifications allow the use of a fourth method of achieving bolt tension. It is the use of special "Alternate Design Bolts," such as the twist-off bolts and the swedge bolts. The specifications give provisions for their installation and inspection.

Installation (Misc.)

Lubrication. Proper lubrication is a critical item. Without lubrication, there simply is no way to achieve a fully pretensioned bolt before it is overstressed in torsional shear (torque). It is recommended by some experts to use dye in the lubricant for ease of inspection.

Reuse of bolts. When the turn-of-nut method is used to induce bolt tension, it often results in tensile stresses that exceed the elastic limit. Also, tests have shown that repeated torquing, loosening, and retorquing reduced bolt ductility. A325 bolts are more ductile than A490 bolts. Accordingly, it is allowed that plain A325 bolts can be reused one or two times but coated A325 and all A490 bolts should not be reused.

Other RCSC installation provisions. The RCSC specifications contain important provisions on handling and storage of fasteners at the jobsite, bolt tension calibration, and acceptable methods of installation. The following are highlights of these provisions.

Handling and storage

Only fasteners that are anticipated to be installed during a work shift should be taken from protected storage. All fasteners that are required to be pretensioned must be kept clean of jobsite rust and dirt. Should rust or dirt be found, it should be thoroughly cleaned and the fasteners should be relubricated before installation.

Inspection Requirements

While the work is in progress on S-C joints, the QC inspector must; (1) determine that all material requirements are met, (2) observe the calibration procedures, (3) inspect the faying surfaces, and (4) monitor all installation procedures. Because it is important to achieve the required tension in S-C and direct tension connections, both a Corps QA representative and the QC inspector are recommended to be present at the time of tensioning and at the time of demonstration testing. A tension measuring device is required at all jobsites where S-C and direct tension connections are being installed. An example of such a device is the Skidmore-Wilhelm Torque-Tension Tester. The device is used to confirm, by demonstration testing, the suitability of the proposed installation method and fastener assembly and to satisfy the minimum pretension force requirements. Also, the Skidmore is used to confirm wrench calibration and demonstrate the understanding and proper use by the bolting crew of the installation method being used.

Summary & Recommendations

This report was prepared to present changes to the RCSC criteria and help achieve a better understanding of HSBC behavior. Quality of a HSBC is improved more by focusing attention on the proper design, installation, and inspection rather than diluting the effort by requiring pretensioning and testing. In summary, the following observations and recommendations are made:

- **S-C connections should be held to a minimum.** This is the most important point in this paper. When installed correctly, S-C connections are very expensive. For typical buildings, S-C connections are generally not needed. For some special connections in buildings, there may be a need for S-C connections. When this is true, only those joints where resistance to slip is critical should be detailed as S-C. For bridges, which are subject to millions of cycles of repeated loading, the use of S-C connections is generally justifiable.

- **B/S connections should be snug-tight.** All connections must be initially tightened to snug-tight and, unless there is a clear serviceability requirement for fully tensioning B/S joints, the extra expenses related to pretensioning B/S bolts are not justified.

- **Prying action on bolts subjected to direct tension**. Since the method used in the AISC manuals does not account for the distortion of the connected parts caused by bending in the fastener assembly, connection details that induce prying action should be avoided.

- **Connections with filler plates.** Connections with fillers must be designed as B/S connections and the slip resistance checked if it is a requirement. Fillers thicker than 1/4 in. must be developed.

- **Bolts in combination with welds.** When bolts and welds are combined, the bolts must be designed against slip to resist their full share of factored load in joint, resulting in the same factor of safety for the bolts and the weld.

- **Fasteners must be well lubricated when installed.** After limiting usage of S-C joints, perhaps the next most important point in this paper is to keep all fastener assemblies, regardless of whether connections are B/S or S-C, well lubricated and

protected. The lubricant preferred by most is "Chem-trend 140 Stick Wax Lubricant" distributed by CASTROL Industries Inc., Chicago, IL.

* **Pretensioning.** S-C connections should be pretensioned as soon as possible. If pretensioning must be delayed, weathered fasteners must be replaced with fresh ones, or field calibration must be conducted on bolt assemblies of the same weathered condition.

* **Inspection.** It is recommended that a Corps QA representative and a Contractor QC inspector be present at all times during calibration, faying surface inspection, and installation of all S-C connections.

* **A325 bolts should be used.** It is recommended that only A325 bolts be used for all HSBC. A307 bolts are not of sufficient strength to qualify as HSBC, and A490 are too strong to be adequately ductile. Note, there are restrictions placed by AISC on A490 bolts in S-C connections, and they should never be used in direct tension.

* **Bolt sizes.** It is recommended that no more than one or two different bolt sizes be used on the same project. The most common sizes are 5/8 to 1-in., with 3/4 and 7/8 in. being the most popular.

- **Identification of tightening requirements.** Bolts in S-C or direct tension connections and bolts in B/S connections which are to be fully tensioned must be clearly identified on the contract drawings.

References

American Institute of Steel Construction. 1986. "Manual of Steel Construction, Load and Resistance Factor Design (LRFD)," first edition, Chicago, IL.

American Institute of Steel Construction. 1988 (Jun). "Load and Resistance Factor Design, Specification for Structural Joints Using ASTM A325 or A490 Bolts," Research Council on Structural Connections, Chicago, IL.

American Institute of Steel Construction. 1989. "Manual of Steel Construction, Allowable Stress Design (ASD)," ninth edition, Chicago, IL.

Kulak, G. L., Fisher, J. W., and Struik, J. H. A. 1987. "Guide to Design Criteria for Bolted and Riveted Joints," second edition, John Wiley & Sons, New York.

The CASE Project

by
Dr. N. Radhakrishnan, PE,[1] *and Paul K. Senter, PE*[2]

Abstract

This paper describes the Computer-Aided Structural Engineering (CASE) project, its objectives, methodology, and accomplishments. The CASE project was funded in Fiscal Year 78 by OCE. The US Army Engineer Waterways Experiment Station (WES) was made responsible for managing the project, conducting research and development, and developing programs using state-of-the-art computer methods. The CASE project has continued to be one of the major projects of the Information Technology Laboratory at WES. CASE project goals are better design/analysis of Corps-type structures, reduction in time required for design/analysis of Corps-type structures, elimination of duplication of program development efforts, organized and cost-effective approach for development of computer programs based on design engineer input, professional engineering analysis and programming, good documentation, and technology transfer.

The total CASE package includes criteria development by task group, survey of available programs, development of new program documentation, review by task groups, field testing of programs, training courses on programs, publication of reports, and Corps-wide release, support, and maintenence.

The CASE project has worked successfully since its inception. Technical task groups are now active in massive concrete structures, steel structures, pile structures and substructures, finite element methods, computer-aided drafting, geotechnical aspects of CASE, building systems, and masonry structures. These groups include 68 design engineers from 26 Corps field offices, 14 engineers from Corps headquarters and laboratories, and representatives from the Navy, Soil Conservation Service, Bureau of Reclamation, and the Federal Energy Regulatory Commission.

The current status and accomplishments of CASE will be described.

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Soil-Structure Interaction Analysis of a U-Frame Lock at Red River Lock and Dam No. 1

by

Dr. Robert M. Ebeling, PE,[1] *and Reed L. Mosher*[1]

Abstract

Lock and Dam No. 1 on the Red River Waterway in Louisiana has experienced a serious siltation problem since its completion in 1983. Sedimentation is deposited during high water against the riverside lock wall at a much higher rate and to a greater level than was anticipated during the design of the project. A recent study considered concepts for a permanent solution to this problem, with a reinforced soil berm having the best potential for a permanent, low maintenance solution. A soil-structure interaction study was conducted at the US Army Engineer Waterways Experiment Station (WES) with the objective of assessing potential lock performance with the construction of a reinforced soil berm adjacent to the riverside lock wall. The soil-structure interaction study was conducted in two phases: (phase 1) confirmation of the finite element model and (phase 2) evaluation of the performance of the proposed reinforced berm with regard to its interaction with the lock and the surrounding foundation soil strata.

The first phase analysis consists of a series of finite element analyses using a nonlinear, incremental construction procedure to model the history of the construction and operation of a U-Frame lock at Red River Lock and Dam No. 1. Good agreement was observed between the results of the finite element analyses compared with instrumentation measurements.

The second phase of the study is an extension of the first phase analysis with the construction of a reinforced soil berm riverside of the lock and subsequent modeling of the operation of the U-frame lock. The analysis showed that the simulation of the construction of a reinforced soil berm adjacent to the riverside lock wall has significant effects on overall lock behavior.

Introduction

Lock and Dam No. 1 on the Red River Waterway in Louisiana has experienced a serious sediment problem since its completion in 1983. Sediments are being deposited during periods of high water at a much higher rate and to a level greater than anticipated during the design of the project. The placement of rock dikes and other hydraulic changes have somewhat alleviated the siltation problem in the lock approaches. However, the sediments deposited against the riverside lock wall have created a very costly maintenance problem.

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In a recent study for the Vicksburg District, the Mobile District considered eight concepts for a permanent solution and concluded that the reinforced soil berm has the best potential because it requires little or no maintenance after its placement. This paper summarizes the findings of an additional study to investigate the potential response of the U-frame lock to construction of the reinforced soil berm and its susequent operation (Ebeling et al. 1991).

The design of a reinforced soil retaining wall on most projects is accomplished using conventional force equilibrium procedures. This method of analysis is sufficient for the evaluation of the stability of proposed reinforcement layout(s) and the evaluation of the overall stability of the reinforced soil retaining wall and its foundation. However, with the placement of a reinforced soil berm adjacent to the lock, these conventional analysis techniques are unable to provide sufficient information to satisfactorily evaluate the performance of the reinforced soil berm with regard to its interaction with the lock and their combined interaction with the surrounding soil. Conventional methods are unable to provide information on deformations which are essential to assessing the performance of the soil berm. To ensure that the berm would accomplish its mission, a soil-structure interaction evaluation was conducted using state-of-the-art nonlinear finite element techniques.

Site

The Red River Lock and Dam No. 1 is the first of a series of locks and dams to provide navigation between the Mississippi River and Shreveport, LA. It is located in the Catahoula Parish in central Louisiana, in a 1.7-mile-long cutoff that shortened the waterway by 8 miles with the elimination of an oxbow meander.

Lock

Construction of Red River Lock and Dam No. 1 was started in 1977 and completed in 1983. The soil founded U-frame lock has an 84- by 785-ft chamber, pintle to pintle, and consists of 18 lock monoliths. Figure 1 shows a cross section through lock monolith no. L-10, located midway along the chamber of the lock. The chamber height of the lock is 71.5 ft, the top of the lock is at el 60.5 ft, and the base of the lock is at el -23 ft. The base slab is 12 ft thick with the elevation of the chamber floor equal to -11 ft. The lock is symmetrical about its center line and each

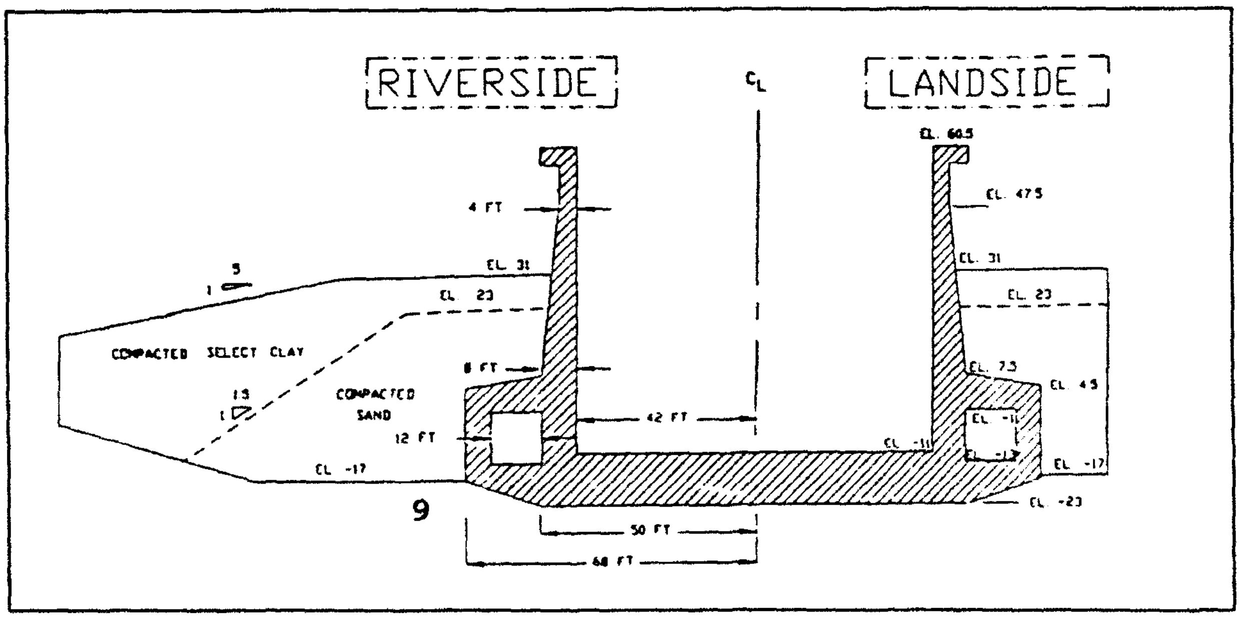


Figure 1. Cross section through lock monolith no. 10

side has a 12-ft-square culvert, formed by an 8-ft-thick interior and 6-ft-thick exterior culvert walls. The tapered stem walls are 8 ft wide at el 7.5 ft, and decrease in thickness to 4 ft at el 47.5 ft. The U-frame structure was constructed with compacted sand and select compacted clay backfill on each side of the culvert stem walls to an elevation of 31 ft. The riverside backfill slopes away from the lock on a 1V:50H slope for a lateral distance equal to 50 ft, beyond which the slope increases to 1V:5H into the new river channel.

Foundation conditions

Geologic sections were developed from the information provided by the 125 general type borings and fourteen 5-in. undisturbed borings made over the site prior to excavation for the new channel and structures. Figure 2a presents a preconstruction geologic section midway along the axis of the lock, corresponding to the approximate location of lock monolith no. L-10. The site is delineated by four distinct soil stratums; the natural levee, the Point Bar Deposit, the Backswamp Deposit, and the sand substratum. The elevation of the ground surface prior to construction was nearly constant at 50 ft, and the delineation between the natural levee and Point Bar Deposit was at approximately el 30 ft. The top elevation of the sand substratum is at approximately -50 ft at the site of the lock.

The deepest deposit is the sand substratum and is comprised of mainly dense sands with some gravel present. The uppermost deposit, the natural levee deposit, is categorized as a fat clay, CH, by its Atterburg Limit values. The Point Bar Deposit is predominantly a silt deposit, with regions of silty sand and poorly graded sand deposits. The Backswamp Deposit consists predominantly of overconsolidated CH clays but also contains interbedded layers of lean clays (CL), silts, silty-sands, and sands.

Analysis Description

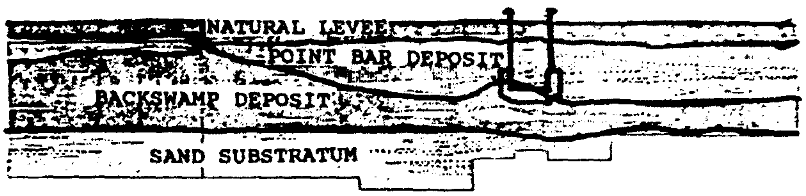
The general purpose, nonlinear, incremental construction, finite element computer program SOILSTRUCT was used to analyze the behavior of the lock and the complex soil-structure interaction by simulating the sequence of lock construction and backfilling, as well as the various water and silt loadings applied to the structure. SOILSTRUCT has the ability to model the nonlinear stress-strain behavior of the soil and allows for relative movement between the soil and structure by using interface elements. Unlike conventional equilibrium procedures, this procedure does not require the use of predetermined force distributions between the soil and the lock but allows for the development of these forces through soil-structure interaction. This procedure of analysis has been successfully used in the past for a wide variety of soil-structure interaction problems and structures, including the evaluation of Port Allen and Old River locks (Clough and Duncan 1969).

Phase 1 analyses

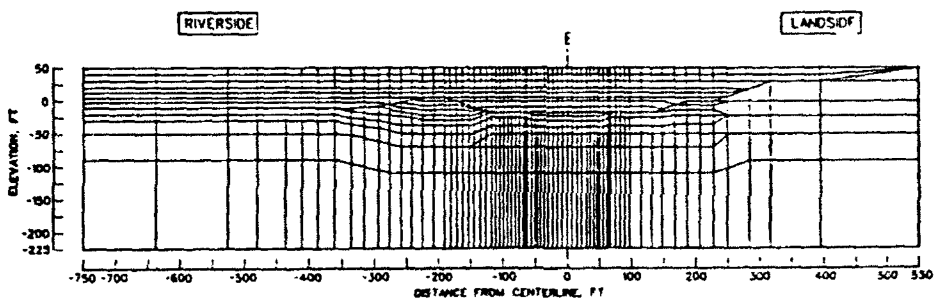
The first phase consists of an evaluation of the behavior of lock monolith no. 11, its backfill and foundation during construction and for three key operational load cases. The stages of construction modeled in the analyses include lowering the water table at the site, excavation, and completion of lock construction and backfilling. The computed results after completion of lock construction and for three key operational load cases for which instrumentation data is available were then analyzed and the results compared to the measured earth pressure measurements.

Dewatering and excavation

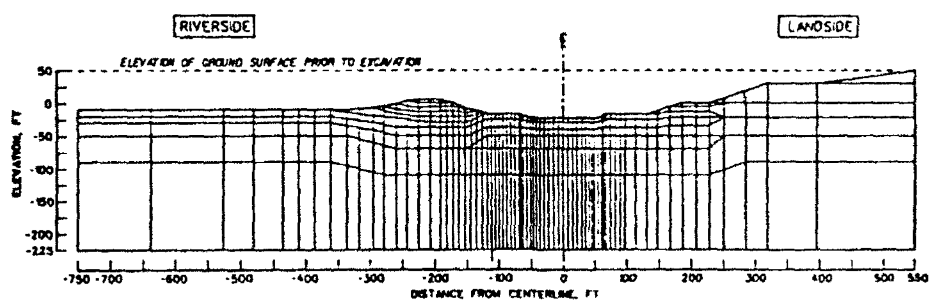
The objective of this first series of analyses was to develop an effective stress regime within the soil foundation that is consistent with that existing in the field after excavation. An additional requirement for the constitutive model of the soil was that the finite elements representing the soil retain the memory of previous maximum values of effective stresses, due to the difference in stress-strain behavior during unloading (and reloading), as compared to primary loading. This affects both the magnitude of horizontal effective stresses computed and the magnitude of future



a. Geologic section

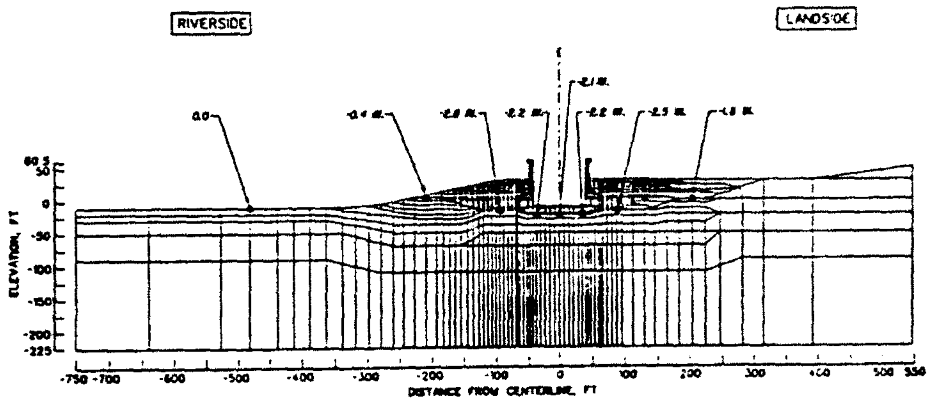


b. Initial mesh



c. Mesh after excavation

Figure 2. Geologic section and finite element meshes used in phase 1 soil to lock interaction analyses (Continued)



d. Displacements after lock construction and backfilling

Figure 2. (Concluded)

computed displacements. The finite element mesh shown in Figures 2b and 2c were used to model the dewatering of the site and excavation. The finite element mesh of the geologic section (Figure 2a) is shown in Figure 2b and models the initial effective stress conditions at the site. The mesh consists of 1,016 two-dimensional elements and 1,081 nodes. It is 1,300 ft long, extending 750 ft riverside from the center line of the lock and 550 ft landside of the center line. The mesh is 275 ft tall with the top of the mesh corresponding to the initial ground surface at elevation 50 ft. The base elevation equals -225 ft and is located within the substratum sands. The initial water table is assigned to el 40 ft.

The initial effective stresses were computed for the soil elements by the gravity turn-on method of analysis incorporated within the program SOILSTRUCT. The hydrostatic water table, initially assigned 10 ft below the ground surface at el 40 ft, was lowered in a series of 10 increments to el -35 ft. The site of the lock and new river channel was excavated in a series of nine stages using SOILSTRUCT with the water table maintained at el -35 ft. The deepest excavation occurs at the site of the lock where the ground surface was lowered 73 ft, from an initial elevation of 50 ft to a final elevation of -23 ft. The final elevation at the new river channel was equal to -10 ft, a 60-ft-deep excavation. The finite element mesh of this cross-section after completion of the excavation is shown in Figure 2c. The removal of overburden during excavation, is not unlike the development of stress regimes within an overconsolidated deposit.

Lock construction and backfilling

Excavation of the site was followed by the construction of the lock and the placement of backfill surrounding the lock. The chronology of lock monolith no. 10 construction and backfilling was followed in the analysis. The finite element mesh of the lock and backfill is shown in Figure 2d. The elements modeling the soil foundation are the same as those in Figure 2c. The mesh consists of 1,152 elements, including 206 elements used to model the lock and a total of 1,257 nodes. The 83.5-ft-tall lock was modeled using 27 rows of elements. The base of the lock was modeled using four rows of elements through the depth of the lock. The stem walls, culvert walls, and the top of the culverts were modeled using pairs of elements. The backfill was modeled using 13 layers of 3.7-ft-high elements. Construction of the lock and placement of the backfill was modeled in 25 load

increments. The water table was maintained at el -35 ft during the course of the construction analyses.

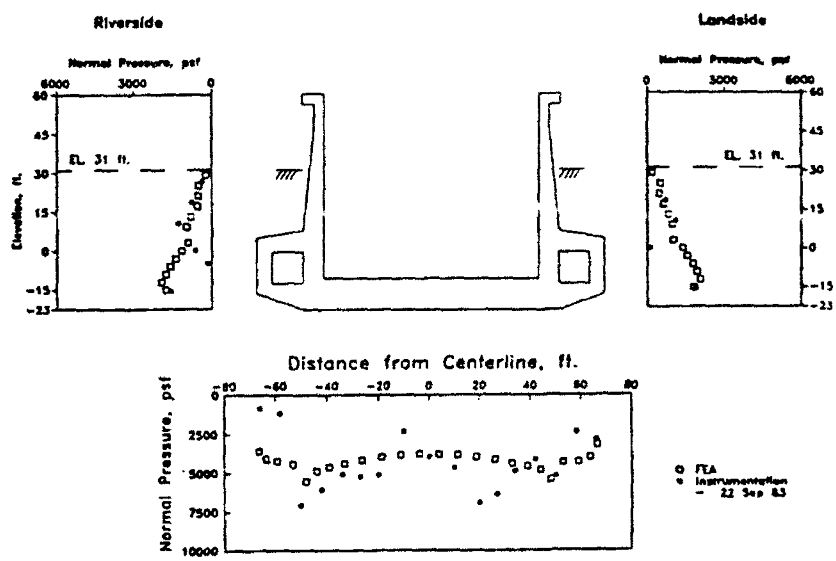
The computed settlement of the foundation, due to lock construction and backfilling, are given in Figure 2d for select nodal points located along the surface of the foundation. The computed settlement below the center line of the lock is equal to 2.1 in. and below the center lines of the riverside and landside backfills equal 2.6 and 2.5 in., respectively.

Figure 3a shows the computed total normal pressures (open squares) computed along the base of the lock and along the culvert walls and stem walls and the 22 September 1983 stress meter measurements (solid circles). No pore pressures were recorded by the Casagrande open-tube piezometers along the base nor within the backfill at this stage of construction. The computed base pressures are symmetrical about the center line and in the shape of an inverted saddle. The largest pressure is computed to be equal to 5,500 psf below the stem walls. Below the center line of the lock the base pressure equals 3,800 psf. The lowest values for the base pressures are computed below the corners of the lock, due to the settlement of the foundation caused by the placement of backfill adjacent to the culverts. The computed normal pressures acting on the walls increase with depth below the surface of the backfill and are equal in magnitude at a given elevation on both sides of the lock. The results from the Carlson PE-50 stress meters are in agreement with the finite element results, considering the trend of the instrumentation measurements and discounting erroneous data.

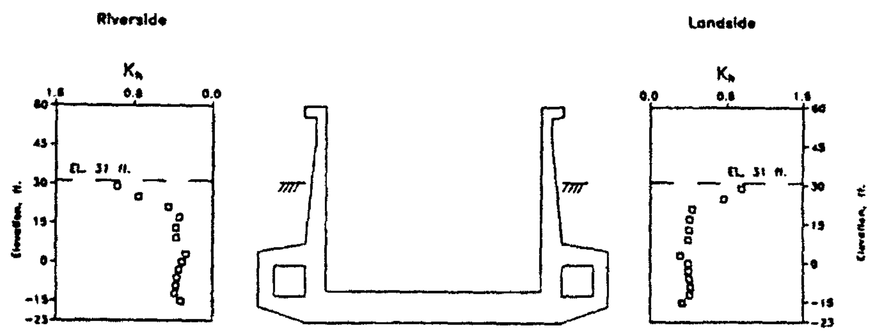
The computed variation in horizontal earth pressure coefficient, K_h, with elevation are shown in Figure 3b after completion of construction. K_h is equal to the ratio of the horizontal effective stress on the lock wall to the effective overburden pressure. The total overburden pressure is computed as the total unit weight of a 1-ft square column of soil above a given elevation. The value of K_h decreases with decreasing elevation along the stem walls and is nearly constant along the culvert walls. K_h ranges from a maximum value equal to 0.98 at el 29 ft to a minimum value equal to 0.34 at el 17 ft along the riverside stem wall. K_h is nearly a constant, averaging 0.35 in value along the riverside culvert wall. The computed K_h distribution along the landside wall is similar to the distribution along the riverside wall, differing by a value of less than 0.05.

The variation in vertical (shear) earth-pressure coefficient, K_v, with elevation are shown in Figure 3c after completion of construction. K_v is the ratio of the vertical shear stress, τ_{xy}, to the effective overburden pressure. A positive K_v value implies that τ_{xy} acts downward along the lock walls. Along the riverside stem wall, K_v increases from a value equal to zero at el 29 ft, to a value equal to 0.13 at el 13 ft. The largest value for K_v is computed to be equal to 0.18 at the top of the riverside culvert wall. The average values for K_v are equal to 0.02 for the 8-ft-thick compacted select clay backfill adjacent to the stem walls and 0.10 and 0.15 for the compacted sand adjacent to the stem walls and culvert walls, respectively.

Figure 3d shows the resulting distribution of factored moments computed within the lock (solid circles) and the limiting values for design moment capacity (solid lines). The moments are computed from the finite element stresses within the lock using the flexure formula. These equivalent moment values are multiplied by a factor equal to 2.21. This factor reflects the extreme load case and is equal to the products of 1.3, the factor applied to hydraulic structures, and 1.7, the factor applied to live loads. The design moment capacity distributions were developed for each of the members comprising the lock using a yield strength of reinforcement steel equal to 60 ksi. As would be expected, the values for the factored moments at all locations within the lock are all well below the

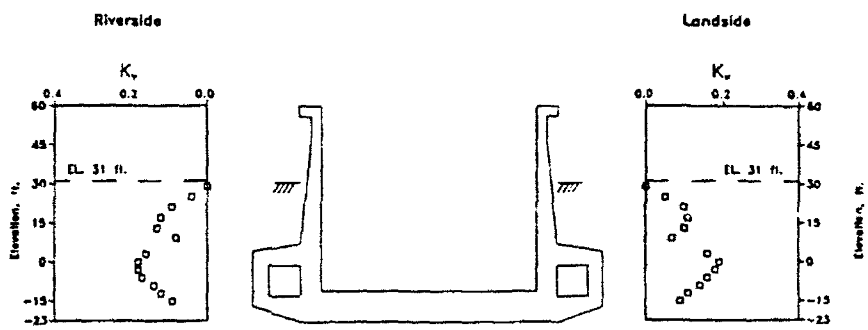


a. Instrumentation and computed results

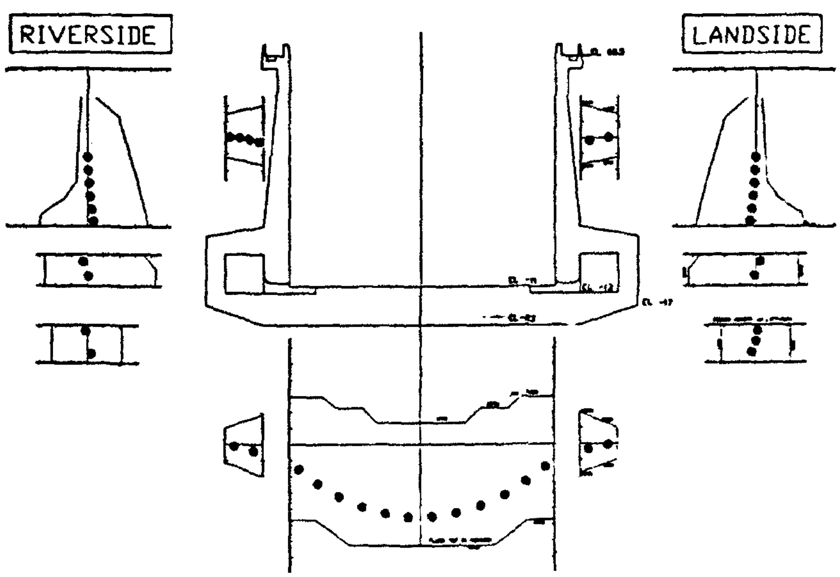


b. Horizontal earth-presssure coefficient

Figure 3. Soil to lock interaction results and measured pressures on 22 September 1983 (Continued)



c. Vertical earth-presssure coefficient



d. Factored moments and limiting moments

Figure 3. (Concluded)

values for the design moment capacity for this loading case.

Operational Load Cases

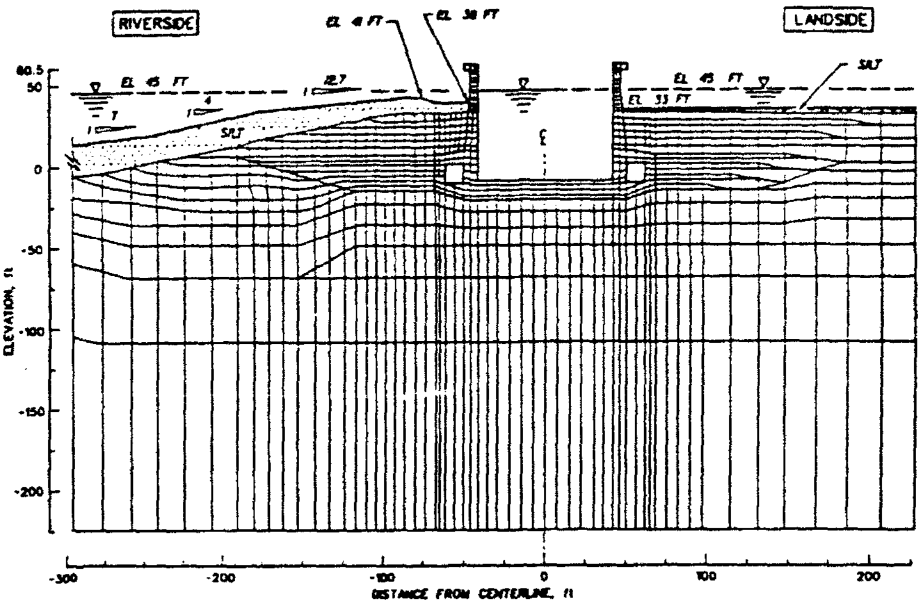
This series of finite element analyses was continued, modeling the flooding of the new river channel, the lock chamber, and subsequent operation of the lock. The river and pool inside the lock chamber and the water table within the foundation of the lock was raised and/or lowered through a series of incremental analyses so that the finite element analyses corresponds to conditions at the lock for three key operational load cases for which instrumentation measurements were available. The three operational load cases consist of a low pool elevation condition (Case 1 occurred on 30 September 1984), a high pool elevation condition (Case 2 occurred on 1 January 1985), and a high pool elevation condition with silt loading against the lock walls (Case 3 occurred on 4 April 1985).

During the high-water period in the spring of 1985, silt was deposited to depths ranging from a few feet to tens of feet against both the riverside lock wall and along the surface of the riverside backfill. The results of the 4 April 1985 US Corps of Engineers survey of the silt depths at lock monolith no. 10 is shown in Figure 4a. On this date, the thickness of the silt adjacent to the lock is equal to 7 ft. The maximum thickness of silt along the top of the backfill is equal to 11 ft at a distance of 40 ft from the face of the stem wall. The elevation of the river above and below the dam was equal to 45 ft. The piezometers within the backfill and immediately below the foundation of lock monolith no. 10 indicate a piezometric head equal to elevations ranging from 41.5 ft to 42.5 ft, with the majority of the measurements equal to 41.5 ft. The response of the lock to the silt loadings for a river and chamber pool elevation equal to 45 ft is shown in Figure 4b.

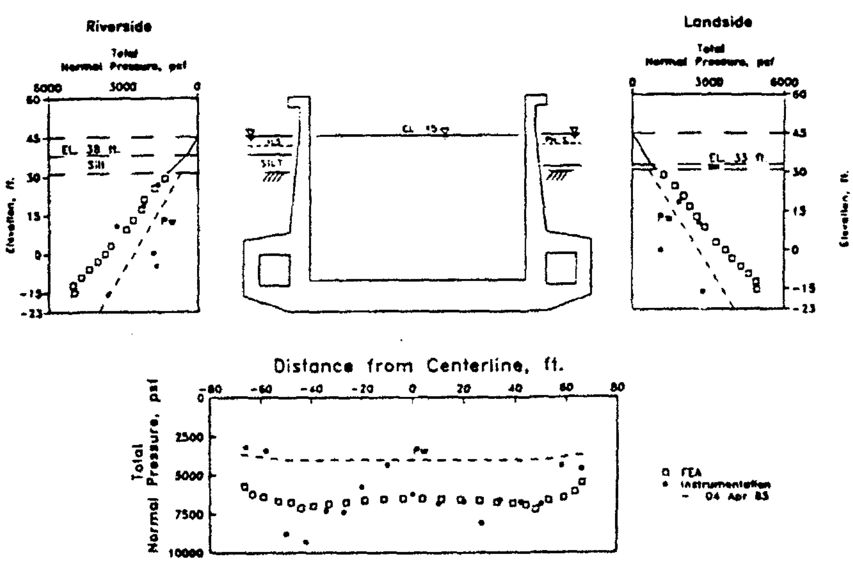
Figure 4b shows the computed total normal pressures (open squares) computed along the base of the lock and along the culvert walls and stem walls and the 4 April 1985 stress meter measurements (solid circles). The computed base pressure distribution shown in this figure is more uniform than the base pressure distribution shown in Figure 3a. The computed maximum values of total normal pressures are equal to 7,100 psf below the riverside stem wall and 7,200 psf below the landside stem wall. Below the center line of the lock the total base pressure is equal to 6,500 psf. The lowest values for the total pressures normal to the base of the lock are computed below the outside corners of the culverts. The base pressure below the riverside and landside culvert differ by 300 psf due to the riverside siltation and are equal to 5,750 psf and 5,450 psf, respectively. The four stress meter measurements along the culvert walls are considered to be erroneous, since their recorded values are less than the corresponding water pressures at those same elevations. The computed results are in agreement with the instrumentation measurements for 4 April 1985. General agreement between the results from the finite element analyses and instrumentation for two additional operational load cases analyses (Cases 1 and 2) was also observed.

Reinforced Soil Berm

The second phase of the study was an extension the first phase analysis with the simulated construction of a proposed reinforced soil berm riverside of the lock, followed by the raising of the upper and lower pool levels, silt loading, and subsequent lowering of the pool levels. Figure 4c depicts the reinforced berm that is used in this series of analyses, as provided by the Vicksburg District. The top of the reinforced berm is at el 45 ft, and the base is at el 13.5 ft. The width of the berm increases with decreasing elevation, increasing from a width equal to 37 ft at the top of the berm to a maximum width equal to 93 ft at el 21 ft. Above el 28 ft, the face slope equals 1.26H:1V. Below el 21 ft, the berm is notched into the riverside backfill and has a face slope equal to that of the existing backfill. The soil comprising the reinforced berm is assumed to be a dense, well drained, select sand.

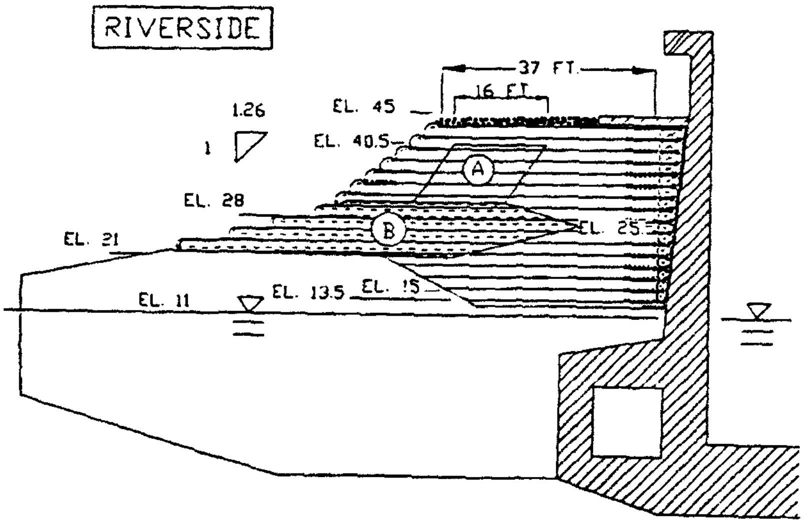


a. Riverside siltation - 4 April 1985

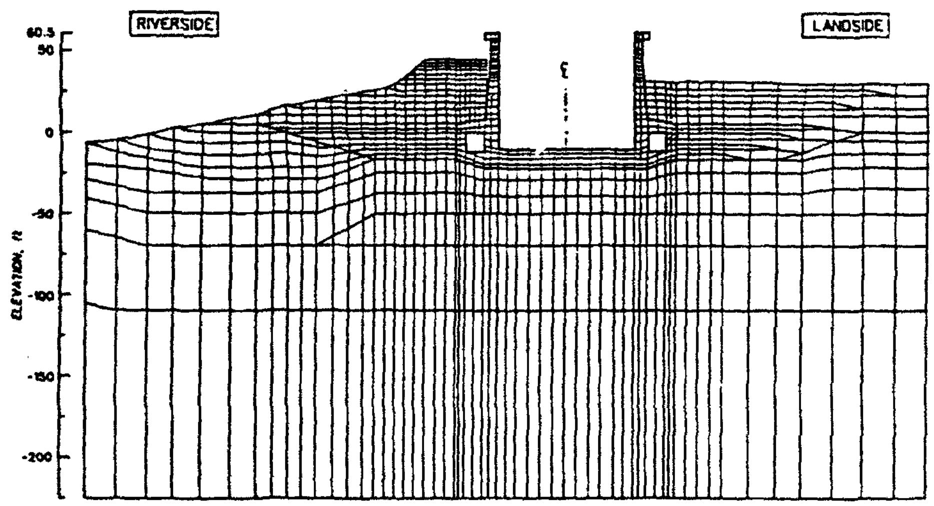


b. Instrumentation and computed results

Figure 4. Soil to lock interaction analysis and reinforced soil berm (Continued)



c. Reinforced soil berm



d. Mesh with reinforced soil berm

Figure 4. (Concluded)

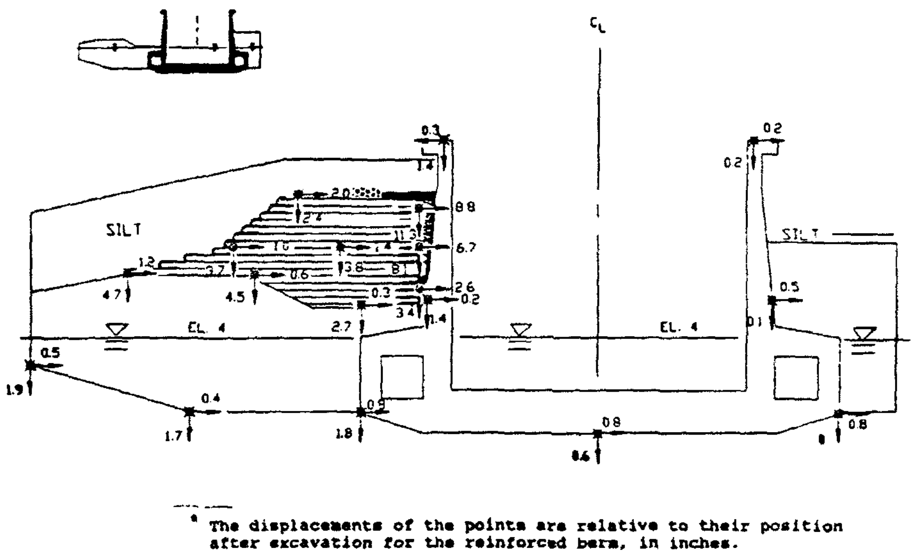
The reinforcement layout used in the analysis varies with location in the berm. The layout of the reinforcement scheme used in the finite element analyses consists of what are referred to in this paper as primary layers and secondary layers of reinforcement. Each primary layer of reinforcement extends across the entire width of the berm. The secondary reinforcement layers are located within the two regions labeled A and B in Figure 4c. Each primary layer of reinforcement extends across the entire width of the berm and are spaced every 1.5 ft in elevation. Six 16-ft-wide, secondary reinforcement layers are placed every 2 ft in elevation between 30.5 and 40.5 ft within the region labeled A. Five secondary reinforcement layers are placed within the region labeled B and are spaced at el 1.5 ft. The finite element mesh shown in Figure 2d was modified within the riverside backfill region to model the reinforced soil berm, as shown in Figure 4d. The reinforcement has the effect of increasing the stiffness of the soil elements comprising the berm and was modeled using one-dimensional bar elements "embedded" within the two-dimensional soil elements. In cases of closely spaced reinforcement layers, the embedment procedure allows for the use of a coarser mesh by eliminating the restriction that reinforcement bar elements must be placed along element boundaries. This formulation was developed by Dr. John Peters of WES for this project (Ebeling et al. 1991).

There is to be a gap between the stem wall and the vertical face of the reinforced soil berm. This region is to be occupied by a geoinclusion to be placed during the construction of the berm. The purpose of the gap is to minimize the effective earth pressures transferred from the reinforced berm to the riverside stem wall between el 13.5 and 45 ft resulting from the lateral deformation of the berm during its construction and resulting from the lateral deformation of the berm in response to water loads and silt loads. The purpose of the geoinclusion in the gap is to fill the gap to prevent the deposition of silt within the gap during periods of high water and eliminate the potential for soil raveling by the reinforced berm.

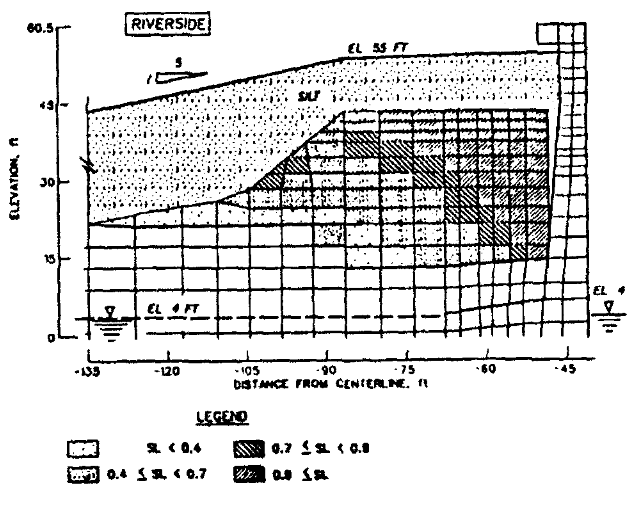
Figure 5a shows the computed displacements at select nodal points, relative to their position prior to construction of the reinforced soil berm. These displacements are computed for after construction of the berm, the raising of the river and pool elevation to the top of the lock, silt loading to el 55 ft, and subsequent lowering of the river and pool to el 4 ft. The settlement distribution varies nonlineally across the base, with the largest value for settlement computed below the corner of the riverside culvert and equal to 1.8 in. The horizontal movements of the base slab and the culverts are less than 1 in. and directed away from the river channel, due to the lateral thrust of the riverside silt deposition. The horizontal movements at the tops of stem walls are less than 1/2 in. and are directed outward, away from lock center line. The horizontal movements of the riverside backfill and the reinforced berm are directed toward the lock.

Figure 5b shows the distribution of mobilized shear strength (SL) within the reinforced soil berm. SL ranges in value between 0 and 1.0. SL equal to 1.0 designates complete mobilization of the shear strength of the soil. This figure shows the shear strength to be fully mobilized within a wedge of the reinforced soil berm located adjacent to the lock wall. Figure 5c shows the distribution of the tensile forces throughout the 28 layers of reinforcement. The largest force is about two-thirds of the long term ultimate capacity and is equal to 2,600 lb per linear ft of reinforcement, computed within the top two layers of the reinforcement.

The values for the factored moments computed within the lock are shown in Figure 5d to be less than the design moment capacity. In previous conventional (equilibrium) design calculations without a reinforced soil berm, the critical location for moments was in the base at the stem wall. In the finite element analysis with 10 ft of siltation on top of the reinforced soil berm, the largest factored moment is at the top of the riverside culvert and adjacent to the inside culvert wall, equal to -1,090 kip-ft, 500 kip-ft less than the design moment capacity.

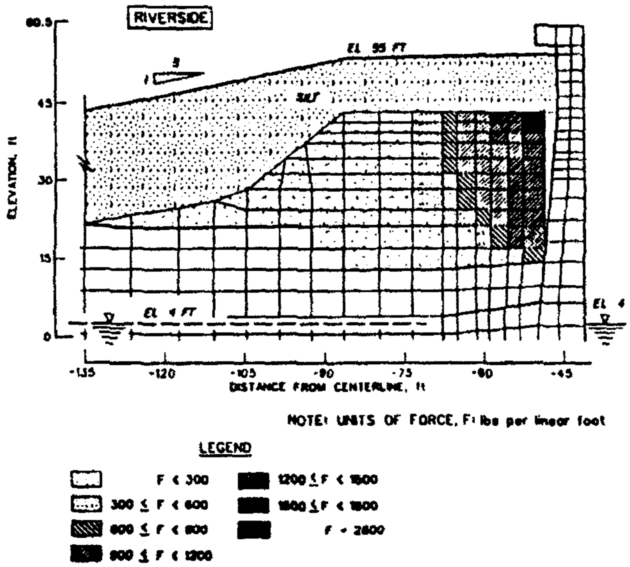


a. Computed relative displacements

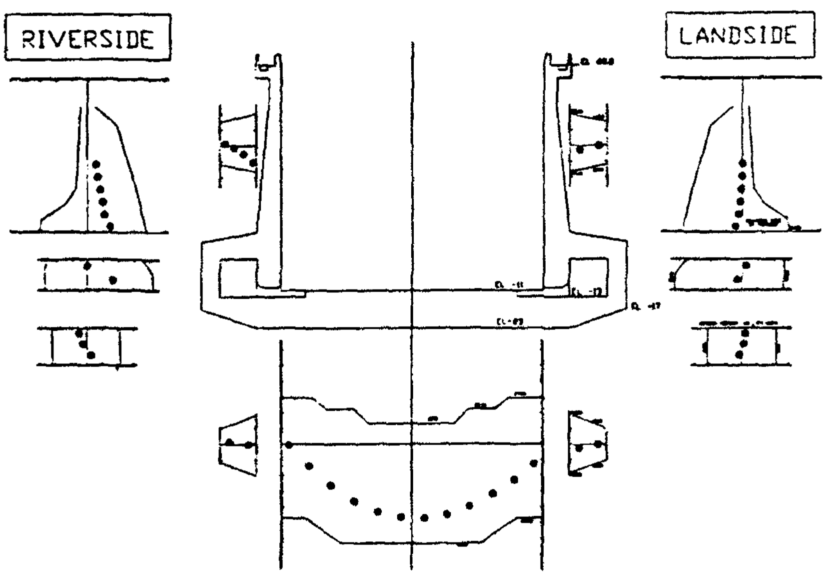


b. Mobilized shear strength in soil berm

Figure 5. Phase 2 soil to lock interaction results for riverside siltation to el 55 ft (Continued)



c. Computed force (lb) in reinforcement



d. Factored moments and limiting moments

Figure 5. (Concluded)

Summary

The four first phase comparisons of earth pressures and base pressures computed using the finite element program SOILSTRUCT to measured earth pressures and base pressures showed good agreement between the results of the finite element analyses when compared with instrumentation measurements. The analyses show the level of soil to lock interaction to be significant with the construction of the reinforced soil berm adjacent to the riverside stem lock wall.

Acknowledgements

This study was funded by the US Army Engineer District, Vicksburg. Significant technical contributions were proved at WES by Dr. John Peters and Kevin Abraham and by C.C. Hamby and Ed Schilling of the Vicksburg District.

References

Clough, G. W., and Duncan, J. M. 1969 (Sep). "Finite Element Analyses of Port Allen and Old River Locks: A Report of an Investigation," Contract Report S-69-3, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ebeling, R. M., Mosher, R. L., Peters, J. F., and Abraham, K. 1991. "Soil-Structure Interaction Study of Red River Lock and Dam No. 1 Subjected To Sediment Loading" (draft report in preparation), US Army Engineer District, Vicksburg, MS.

Recent Developments in the Study of the Behavior of Retaining Walls

by

Reed L. Mosher[1]

Abstract

Over the last four years a research effort has been under way at the Waterways Experiment Station to investigate the fundamental behavior of gravity-retaining walls. The intention of this paper is to update the Corps' structural engineering community on the findings of this research effort.

Since the last structural engineering conference a number of new studies have been completed. These included an experimental study of earth pressures on retaining structures and an analytical study on the behavior of soil rounded retaining walls. A number of significant findings have resulted from these investigations. The paper summarizes these studies and their findings in light of future design considerations.

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Finite Element Analysis of Anchored Sheet Pile Bulkhead

by

Kevin Abraham[1] *and Reed L. Mosher*[2]

Abstract

This paper presents a finite element study of an instrumented anchored bulkhead constructed at the Port of Toledo, Ohio. The purpose of the study was to investigate the behavior of anchored bulkheads and to clarify some of the abnormalities found in the classical design/analysis procedures for anchored bulkheads. The bulkhead system investigated consisted of a sheet pile front wall tied to an anchor wall. The interaction behavior between the walls was examined.

The analyses were performed utilizing a nonlinear finite element code. The finite element code employed has the capability to simulate the construction process of bulkhead systems. A complete construction simulation is necessary to account for stress path dependency and nonlinear behavior of soil.

The finite element studies provided information on deformation patterns for the configuration, effects of sheet pile penetration on the stability of the system, and effects of the wall and anchor stiffness on the soil stresses on displacements. These comprehensive analyses identified critical factors essential for improved design/analysis procedures for anchored sheet pile bulkheads.

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Mill Creek, Ohio, LFPP - Section 1
Post-and-Panel Wall With Anchors - Lessons Learned

by

Daniel E. Beyke, PE[1]

Abstract

Tied-back post-and-panel type retaining walls have been used in previously constructed sections of the Mill Creek, Ohio, Local Flood Protection Project (LFPP). Therefore, this system was chosen for the retaining wall which was required in Section 1 of the Mill Creek LFPP. This post-and-panel wall consists of steel H-piles or W-sections as posts, cast in 3-foot (0.91m) diameter concrete caissons spaced at 6 feet (1.83m) center-to-center, with the caissons drilled into rock, and with precast concrete panels between the steel posts. The posts are designed as a vertical beam which is supported horizontally by the rock at the bottom and by wales as necessary at the top. The wales are supported by rock anchors spaced at either 6 feet (1.83m) or 12 feet (3.66m) center-to-center. Because of the loads on the walls, high-capacity strand anchors were chosen for use in supporting the top portion of the walls. These anchors ranged in size from a 6-strand anchor with a design load of 246 kips (1,094kn) to 12-strand anchors with a design load of 492 kips (2,188kn).

The capacity of the 12-strand anchors was much higher than that of the anchors used on the post-and-panel walls installed in the previously constructed sections of the project. Therefore, even though we had some previous experience with anchored walls, due to the high capacity anchors, we experienced some difficulties with the design and construction of these walls which we had not previously encountered. Such difficulties included the deflection of the pile and the cracking of the lagging during anchor testing, problems in the interpretation of the test readings, the optimization of the anchor spacing, etc. At the same time, the contractor encountered an existing concrete retaining wall which interfered with the installation of the new wall and necessitated a modification in a portion of our wall design.

These problems, some of which should have been anticipated and others which could not have been foreseen, were all solved in an expedient manner, with a very serviceable product as a result. We also learned some valuable lessons to be used in future projects.

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Introduction

The Mill Creek, Ohio, Local Flood Protection Project is located in Hamilton County in Southwestern Ohio. The Mill Creek has a drainage area of 165 square miles (427 sq.km.) and is 28 miles (45 km.) long from its headwater to its confluence with the Ohio River in western Cincinnati, Ohio. The creek parallels Interstate 75 (I-75), also called the Mill Creek Expressway. The creek also parallels Conrail's North and South mainline railroad tracks through Cincinnati.

Section 1 of this project is located between stream miles 1.6 and 2.3 of the Mill Creek with the downstream end being just downstream of the Western Hills Viaduct and the upstream end being just downstream of the Hopple Street Viaduct. By widening and/or deepening the channel, this project will provide a 50-year level of protection for the affected area. Types of channel bank treatment provided include riprapped slopes, concrete-paved slopes, and tied-back post-and-panel type retaining wall.

A retaining wall was required for 2,490 feet (760m) of the right bank of the creek to provide the required flow area and to protect the existing railroad which ran along the top of the slope of the right creek bank. Hoping to make use of the design calculations and drawings from previous sections of the project, which would cut the design time and cost, the designers chose to use a tied-back post-and-panel wall system for the retaining wall. This type of wall was well suited for our conditions also in that it allowed for the wall installation without interfering with the railroad. Also, the preliminary boring information showed that the underlying rock was relatively shallow, thus reducing the required length of the anchors, which is the most costly part of this type of wall.

Design of Wall

When the geological investigation and testing was concluded, their findings showed that there was a weakened zone at the interface between the overburden soil and the rock beneath. Also, the surface of the rock was sloping toward the stream. Taking into consideration that there has been a history of slides in the area of the creek, our Geotechnical engineers computed driving forces upon the wall which were greatly in excess of those on any other section of wall on the project. Therefore, we were unable to use the previous designs as we had hoped. Although we realized that the design time would be increased since we could not use the previous wall designs and "site-adapt" them, we did not think that this would affect the wall construction.

Our design calculations indicated that, depending upon the location and depth of rock, our anchors would range in size from a 6-strand anchor with a design load of 246 kips (1,094kn) to a 12-strand anchor with a design load of 492 kips (2,488kn). The 12-strand anchors had nearly two times the design load of any anchors previously designed for retaining walls used at the Mill Creek project. However, upon discussion with contractors who have installed anchors we found out that this was not outside the limits of the anchor size which could be feasibly installed at 6-foot (1.83) centers. Therefore, we proceeded with the preparation of plans and specifications.

Construction of Wall

The construction problems began when the contractor began to do the performance tests on the anchors. The district office got a call from the Contracting Officer (CO) telling us that the 12-strand anchors were failing the tests and wanted to know what to do. As it turned out, this problem was the easiest one to solve because it solved itself. Let me explain.

For all the previously installed anchors, the measured creep during the performance tests was a small percentage of that allowable in our specification, which is 0.080 inches (0.202cm) during the final time increment of testing. When we examined the test readings from the contractor, we discovered that although the measured creep was close to the allowable, it did not exceed it. Therefore, this problem was not a problem at all. What had

happened was a combination of two things: first, our CO was so accustomed to the very small creep measurements of the smaller anchors that when these larger measurements were recorded he thought that the anchors were slipping. The second factor was that the contractor, though experienced in anchor installation, was accustomed to using the FHA specifications, which differed from ours in that it only allows the creep to be 0.080 inches (0.202cm) over the total measuring period, rather than the last time increment. Therefore, he actually thought that the anchors were failing but, according to our criteria, they were acceptable.

The testing of the 12-strand anchors led to another problem which was not anticipated. In the process of the stressing of the top row of anchors, the posts would deflect more than an inch. This in itself was not a problem but when the anchor at one post was stressed, the relative deflection of that post with the adjacent post would cause the concrete lagging to crack at the ends. We checked our lagging design again which turned out to be adequate. The CO checked with the lagging manufacturer to make sure that the lagging was constructed properly which it was. At this point we were at a loss as to what we should do.

The credit for finding the solution to this problem belongs to the CO. He surmised that the differential movement in the adjacent posts was putting torsion into the lagging for which they had not been designed. This was due to the fact that the lagging was tack-welded to the flanges as they were placed to hold them in place until the backfill was placed behind them. This tack-welding essentially made a moment connection at the ends of the lagging panels where they were meant to be simply supported. Thus, when one post would deflect in relation to the adjacent post, since the panel was not free to rotate, they would crack. Therefore, the CO ordered the contractor to cut through the tack weld of the lagging just previous to the testing of the anchors. This took care of the cracking of the lagging.

At the same time as these problems were being discussed the really big problem arose. We were aware of some existing concrete struts which were in the bottom of the creek which would interfere with the placement of the concrete pavement which was to serve as our stream bottom. However, when the contractor began to excavate in the area of these concrete struts, he uncovered a massive concrete retaining wall which was just streamward of our wall. This existing wall was about 900 feet (275m) long, with the top at about the elevation of the top row of our anchors. By the time that we discovered that the wall was there, and the size of it, the contractor had already placed about 90 percent of his posts. Therefore, we could not realign our wall without great expense. Also, the wall was so close to our proposed wall that the contractor could not place the bottom row of anchors and, in some places, he could not place his top row. We considered removing this wall and in fact the contractor began to remove the end of it to be able to place his last few posts. This is when we discovered that the wall was reinforced with railroad rails at about 4.0 feet (1.22m) on center both horizontally and vertically. Therefore, the cost for removing the wall was also exorbitant.

After further investigation we discovered that the struts were apparently placed to support the existing concrete retaining wall. These struts were also shown to be keyed into the rock at the bottom of the stream. We reanalyzed the post-and-panel wall with the existing wall left in place and found that by installing the top row of anchors the wall would be adequately stable. This took into account the passive pressure from the existing concrete wall and the struts holding it at the base. We then filled in the space between the existing and the new wall with concrete. This not only allowed the existing wall to stay in place but also provided some savings since the bottom row of anchors were not needed.

Conclusions

There were several lessons that were learned in the process of solving these problems. The first lesson was that whenever a former design is to be site-adapted to another location, make sure that you compare the conditions of the new site with those of the site of the previously designed structure. Even though the structures may be in the same general area, the design conditions may vary greatly.

A second lesson learned was the importance of looking for as-built drawings of existing structures early in the design phase. A little extra effort early on can save you a lot of time and money down the road.

Finally, and probably most importantly, we learned how important it is for the Engineering Division and the Construction Division to work together in every phase of a project. Good communication is essential between Engineering and Construction, and had we not previously established a good working relationship between the design team and the construction personnel, especially the field personnel, we could not have resolved these problems without much more time lost and money spent.

References

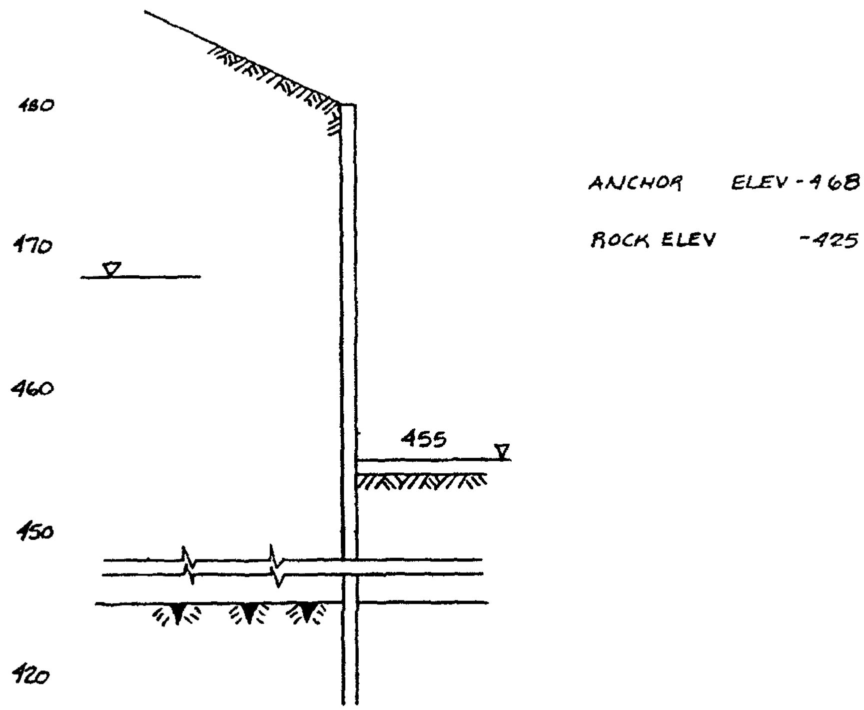
Bethlehem Steel, Inc. 1988 (Jun). "Steel Sheet Piling Handbook."

Nicholson Anchorage Division of Nicholson Construction Company. 1984. "Rock and Soil Anchor Manual."

Pile Buck, Inc. 1987. "Steel Sheet Piling Design Manual."

Schnabel, Harry, Jr. 1982. *Tiebacks in Foundation Engineering and Construction,* McGraw-Hill.

US Department of Transportation. 1982 (Jul). Federal Highway Administration "Tiebacks," Final Report, Report No. FHWA/RD-82/047, Offices of Research and Development, Washington, DC.

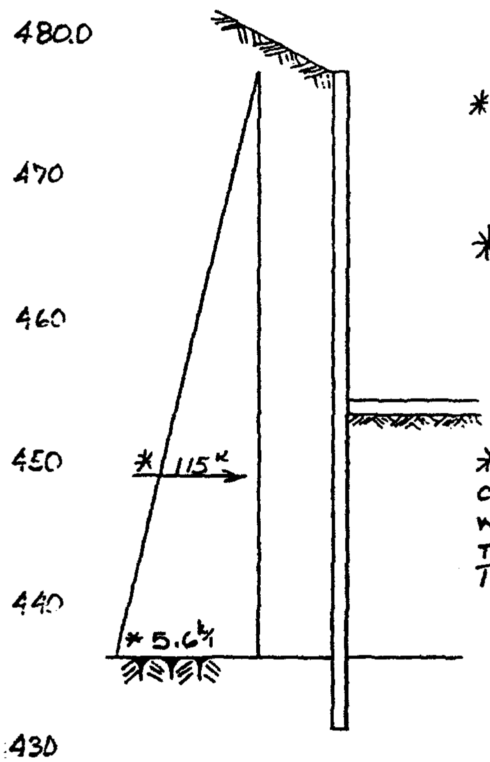


Based on Geotechnical Calculations (PAGE 1G) the depth of wall below channel invert = 30.25' (say 30'). for FS. = 1. This "length of wall" = 56' will be used to design post and panel wall, anchors, wale and concrete lagging. RANKINE Theory was used to calculate pressure distribution.

CFRAME RESULTS (RUNDATE = 89/06/12 RUN TIME H.54.58)

HORIZ. ANCHOR FORCE = 24.1 k/LF OF WALL (EL 468)
ROCK REACTION = - 3.7 k/1
MAX. PILE MOMENT = 1,785 'k/1
MAX. PILE SHEAR = 19.8 k/1

Figure 1. Sta. 1113+50 to Sta. 1117+28, right bank



THIS SECTION AND THE FOLLOWING INFORMATION PROVIDED BY ED-G

*F.S. = 1.0

TOP OF WALL EL. 478.0

WATER EL. 467.0

CHANNEL BOTTOM EL. 455±

TOP OF UNWEATHERED ROCK = 437± *

*DRIVING FORCE = 115^K (HORIZ) WHICH INCLUDE: EARTH AND WATER.

AT-REST EARTH PRESSURE = 0.035 ^{K}/1/1

* ACTUAL LOAD FOR F.S. = 1.0 IS 103^K AS COMPUTED BY ORLED-G. UPON DISCUSSION WITH ORD, ORLED-G PERSONNEL INCREASED THE MAGNITUDE OF THE FORCE TO 115^K. THEREFORE F.S. IS ACTUALLY GREATER.

*Load at base provided by Geotech = 5.6^k/1

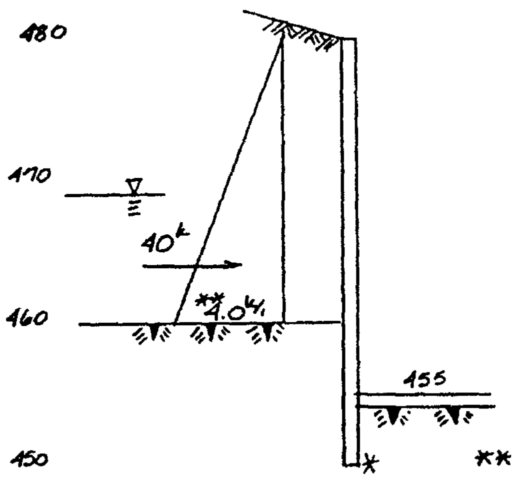
Load at base after adjusting to trapezoid = 1.88^k/1

RESULTS FROM CFRAME-X0030 (FILE = RB11200)

HORIZ. TOP ANCHOR FORCE	= 27.5 K/L.F. OF WALL	AT EL. 468.0
HORIZ BOTTOM ANCHOR FORCE	= 54.5 K/L.F. OF WALL	AT EL. 457.0
ROCK REACTION	= 20.9 K/L.F. OF WALL	
MAX. PILE MOMENT	= 1,437 "K/LF OF WALL	
MAX. PILE SHEAR	= 34.1 K/L.F. OF WALL	

*ED-G SECTION SHOWS ROCK @ 450, BUT THEIR DRIVING FORCES WERE COMPUTED FOR ROCK @ 437

Figure 2. Sta. 1120+00 to Sta. 1124+00, right bank



THIS SECTION AND THE FOLLOWING INFORMATION PROVIDED BY GEOTECH. (P.46)

F.S. = 1.0
TOP OF WALL = 480
WATER EL = 469
CHANNEL BOTTOM = 455
TOP OF UNWEATHERED ROCK = 460
40^k INCLUDES EARTH & WATER FORCES

Subtracting out water from 40^k load:

$$40^k - (469-460)^2 \tfrac{1}{2}(0.0625 \, k/ft^3)(1')$$

$$40^k - 2.53^k = 37.46^k \text{ earth force only}$$

** Load at base provided by Geotech = 4.0 k/l
Load at base after adjusting for trapezoid = 2.64 k/l

*Revised elevation of assumed pile fixity from 7' into rock to 5' into rock (at ORD meetin

RESULTS FROM CFRAME (RUN DATE: 89/07/11 RUN TIME: 07.55.10)

		Revised Load Distribution	Previous Load Distrib
(EL 470)	HORIZONTAL ANCHOR FORCE	= 37.11 k/LF OF WALL	44.56 k/LF
	"ROCK REACTION"	= 10.85 k/LF	17.46 k/LF
	MAX. PILE MOMENT	= 815.4 'k/LF	1063 'k/LF
	MAX. PILE SHEAR	= 20.5 k/LF	29 k/LF

PREVIOUS ANCHOR SIZE	= 13 STRAND
ANCHOR SIZE BASED ON REVISED LOAD ONLY	= 11 STRAND
ANCHOR SIZE BASED ON REVISED LOAD & METHOD OF ANALYSIS	= 8 STRAND

RESULTS OF REVISED ANALYSIS & DESIGN		RESULTS OF PREVIOUS ANALYSIS & DESIGN	
PILES	- W 21 x 101	PILES	- W 21 x 132
ANCHORS	- 8 STRAND	ANCHORS	- 13 STRAND
WALES	- 2-W 18x97	WALES	- 2-W 18x97

Figure 3. Sta. 1133+00 to Sta. 1138+40, right bank

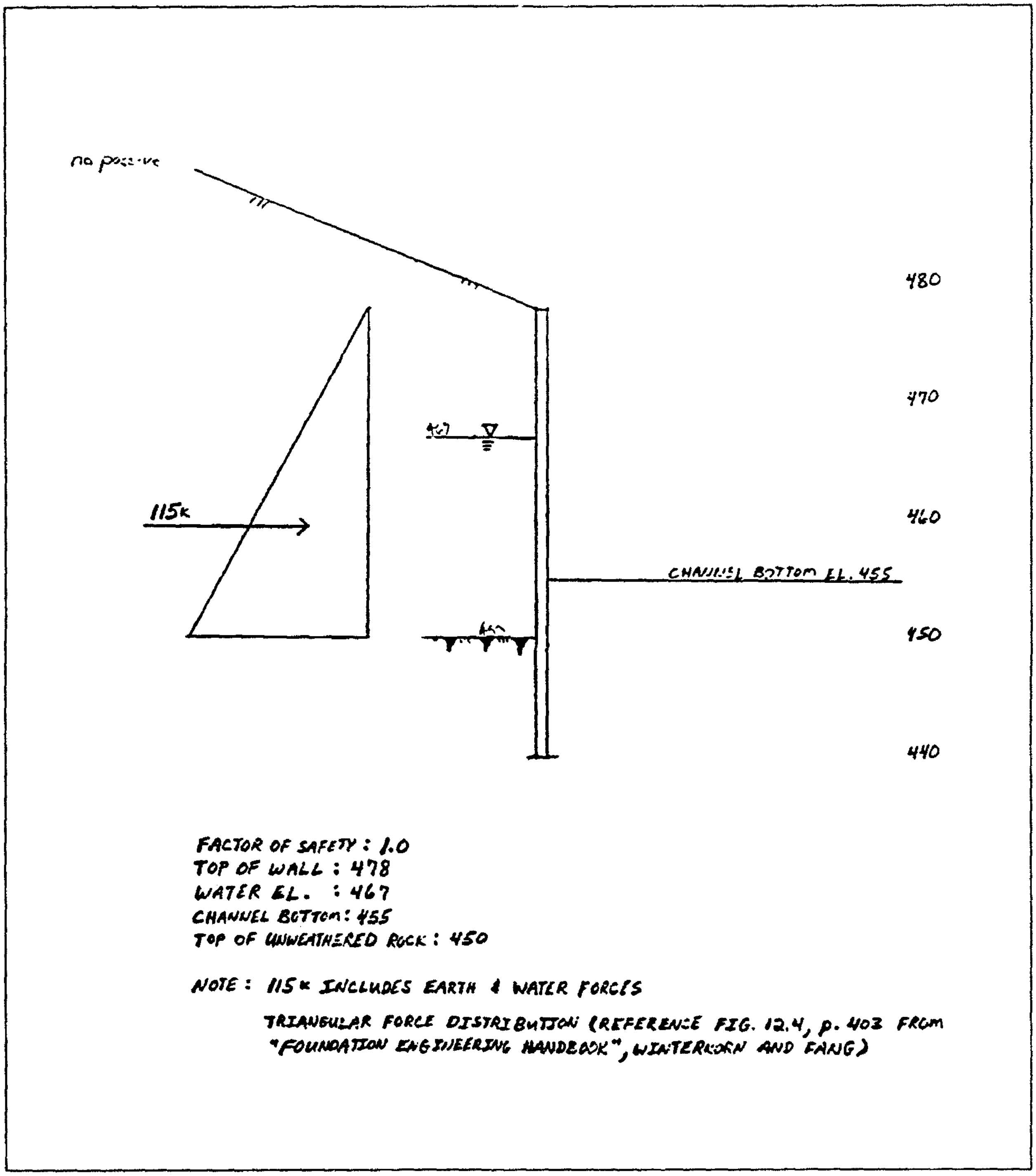
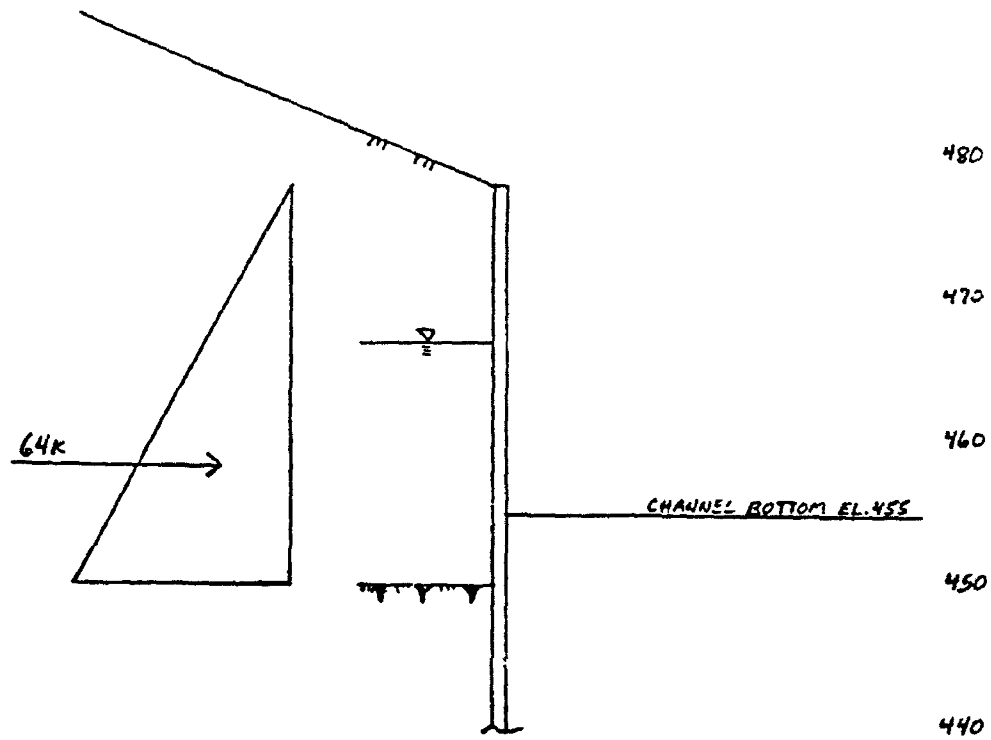


Figure 4. Mill Creek, Ohio, Section 1, tieback wall forces

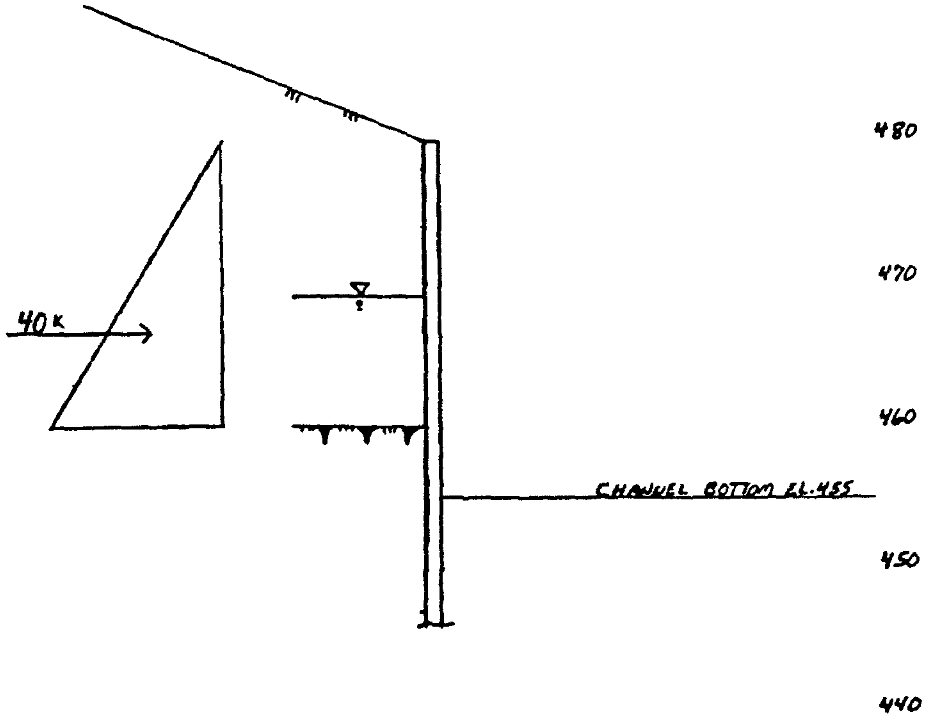


FACTOR OF SAFETY : 1.0
TOP OF WALL : 478
WATER EL. : 467
CHANNEL BOTTOM : 455
TOP OF UNWEATHERED ROCK : 450

NOTE : 64K INCLUDES EARTH & WATER FORCES

TRIANGULAR FORCE DISTRIBUTION (REFERENCE FIG. 12.4, p. 403 FROM "FOUNDATION ENGINEERING HANDBOOK", WINTERKORN AND FANG)

Figure 5. Mill Creek, Ohio, Section 1, tieback wall forces



FACTOR OF SAFETY : 1.0
TOP OF WALL : 480
WATER EL. : 469
CHANNEL BOTTOM : 455
TOP OF UNWEATHERED ROCK : 460

NOTE : 40K INCLUDES EARTH & WATER FORCES
TRIANGULAR FORCE DISTRIBUTION (REFERENCE FIG. 12.4, p. 403 FROM "FOUNDATION ENGINEERING HANDBOOK", WINTERKORN AND FANG)

Figure 6. Sta. 1133+00 to Sta. 1138+40, right bank

Bulkhead Design and Construction Problems at Engineering Yard, Savannah River Savannah, Georgia

by
Kirti S. Joshi, PE[1]

Bulkheads to provide support to docking facilities are becoming more frequent with the tonnage growth in the Savannah port.

Design of wharfs along with bulkhead is getting more involved, and the art is slowly being perfected to suit the needs of the client. This paper presents the Lessons Learned from the design and construction of the Bulkhead at the Engineering Yard, Hutchinson Island, Savannah, Georgia.

The City of Savannah will be one of the host cities for the 1996 Olympic games. This fact, combined with the growth of the last ten years in the tonnage handled at the port facility, has compounded the problems for bulkhead design. The facility to be built is located at the Engineering Yard used by Savannah District. It will have 300 ft of docking area and 300 ft of bulkhead. The docking area is intended to service dredges, survey boats, jack-up barges, and other miscellaneous vessels. Erosion from wake waves of the passing ships will be considered.

The problems encountered during design and construction include:

- Errant vessels hitting the bulkhead and moored vessels.
- Corrosion study.
- Aesthetics.
- Increased dredge depth of the Savannah River.
- Archeological study.
- Pile driving in the area known to have buried debris.

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Channel Wall Study for the River Des Peres Flood Control Project

by
Rochelle R. Ross[1]

Abstract

Through the years, the Corps of Engineers' way of doing business has changed. We are now involved with more cost-sharing projects, and therefore, there is a greater demand to construct cost-effective designs. This in turn forces us to keep up with progressing technology and find new and better solutions to old problems.

One of the Corps of Engineers' old and continuing problems is flood control through highly populated residential and commercial areas with limited space for construction. The River des Peres Flood Control project is such a problem.

A channel wall constructed of gabions was presented in the Reconnaissance Report as the method to use. But, because of the excessive cost to the Corps of Engineers and the public, it was decided that a less expensive wall needed to be found in order to increase the benefit/cost ratio to an acceptable value.

This paper will present various innovative methods researched and evaluated for the construction of a flood control wall along the River des Peres in St. Louis, MO.

Introduction

I am sure you all know that the Corps of Engineers cannot do business as they once did many years ago. Today we must be concerned with the number of hours and the amount of money we spend. With the changing times, we have more cost-shared projects which force us to construct the least expensive, most effective structures. Like everyone else, we and our customers want the best for the least. The River des Peres channel improvement project is such a cost-shared project.

The River des Peres enters the Mississippi River at river mile 172.1, which is approximately 23 miles downstream from the confluence of the Mississippi and Missouri Rivers. The River des Peres watershed covers 111 square miles and includes portions of unincorporated St. Louis County and the city of St. Louis and all or parts of 42 municipalities within St. Louis County. This project is divided into two sections, Deer Creek and University City. The walls researched were for a 0.38-mile portion of the University City.

Constraints

Like all projects, there are many problems and constraints one has to deal with and overcome. These constraints include construction limits, the cost of the materials and construction, scour and frost protection, construction right-of-way, the height of the wall, the width of the channel, and others such as the soil

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properties. For this particular project, the height of the wall was set at 15 ft, and the channel width varied depending upon the roughness of the wall face. For a smooth surface, such as concrete, the required width was 55 ft, and for a rough surface, such as gabions, the required width was 65 ft. Based on scour and the frost line, the bottom of the footing needs to be 3 ft below the creek bed. The construction limits are restricted by existing residential and commercial buildings. The top of rock is located within 5 ft from the creek bed. And of course, the cost must be inexpensive, and the benefit/cost ratio must be an acceptable value.

Walls Researched

There are four classes of wall systems: gravity, cantilever, anchored, and mechanically stabilized backfill. Gravity walls rely on the weight of the wall system to resist overturning. Most consisted of precast concrete and earth or rock, and one was made of wire baskets filled with rock (i.e. gabions). The cantilever wall is designed such that the stem, heel, and toe each acts as a cantilever beam. Anchored walls are held vertical by anchors penetrating deep into the backfill. This type of wall system was not looked at for this project. Mechanically stabilized backfill stems from the familiar reinforced earth wall, in which steel strips interact with the backfill to form a coherent mass of reinforced earth, behaving in some ways like steel-reinforced concrete. Because of the location of the rock, sheet-pile walls were not considered.

The design procedures were similar for all types of walls looked at. The procedures consisted of bearing capacity, sliding, and overturning and were done using the Corps of Engineers' computer program CSLIDE and hand calculations following Corps of Engineers and Naval Facilities criteria. Initially, a typical cross section through University City was taken, and each wall layout was superimposed onto it. Quantities of excavation, backfill, premanufactured materials, and other items which depend on the type of wall were found for each wall type, and cost comparisons were made.

The five walls researched consisted of the following: (a) Armco Bin-Type Walls, (b) Doublewall, (3) Mechanically Stabilized Earth, (d) TechWall, and (e) WaterLoffels. Each of these is classified as one of the above wall systems and is described in detail below.

Armco Bin Walls - Contech

Armco Bin-Type Retaining Walls are a system of adjoining closed-face bins, each 10 ft long. They consist of sturdy, lightweight steel members that are bolted together at the jobsite. For this particular job, precast concrete panels were used instead of the steel for the front facing. These slide down slots located on both side steel members. Once they are filled with soil, they are transformed into a gravity-type retaining wall.

Since all four sides of each cell are composed of overlapping steel members and precast concrete facing panels, the fill is easily contained within the structure. Armco Bin-Walls have the ability to withstand temperature variations and the effects of ice and snow. The steel members are supplied in galvanized material to help prevent against corrosion.

This particular design called for a 12-ft base. The fill used in these bins is pervious. A filter fabric lines the front facing panels to aid in drainage and in the containment of the fill. Because this is a gravity-type wall, support for the wall is needed under the earth mass. On rigid foundations, provision must be made to allow slight settlement of the vertical corner members. This is usually done by providing a compressible cushion under the grade plates with approximately 8-in. of loose fill.

Contech's quoted cost for the Bin Wall was $20.00/sq ft of wall constructed without excavation. The Corps of Engineers' estimate for the construction of the Armco Bin Walls was $1,800,000.

Doublewall

Doublewall is a gravity retaining wall system. This system consists of precast, interlocking,

reinforced concrete modules which vary in size depending upon the application. Each module consists of two face panels held rigid and apart by connecting beams. Once in place, the units are filled with earth or with screened or crushed stone, depending upon the application, to form a gravity retaining wall.

Because this project does not involve wave action from fluctuating water levels, the units would be filled with earth fill. To aid in drainage, the front vertical joints are sealed with filter fabric. A rubber pad is placed between horizontal joints. Other than the horizontal and vertical joints, there are no openings in the face of the concrete wall. This provides protection against material loss.

Construction of Doublewall consists of pouring a concrete footing at the toe of the structure, lifting the modules off the truck and placing them by a crane, filling them with pervious fill (10 percent passing a No. 200 sieve), and compacting the fill, then repeating the process (excluding pouring the footing) until the desired height is reached. According to Doublewall, the installation rate is approximately 2,000 sq ft/day, using a crew of five.

Corrosion problems are eliminated since the system does not require forms, bolts, nuts, pins, fasteners, or special strips under layered embankments. The wall is vertically flexible so that differential settlements are tolerable, although it may affect the appearance. Because this type of wall system depends upon its weight for stability, utilities may be installed behind the wall without affecting it.

This particular wall required a 10-ft base. Compared with the Armco Bin-Walls, the excavation cost is 15 percent less. The quoted price from DoubleWall Corporation for the River des Peres application was $27.00/sq ft of wall, which includes the cost of the modules and footing in place. This is considerably more than the materials used for the Armco Bin-Wall. The Corps of Engineers' estimate for the construction of DoubleWall was $1,800,000, the same as Armco Bin-Walls.

Mechanically Stabilized Backfill

The mechanically stabilized backfill that was researched for the River des Peres project is manufactured by the Reinforced Earth Company. The Reinforced Earth structure is a single, coherent gravity mass that can be engineered for specific load requirements. The interlacing of soil and reinforcements develops friction at the points of contact between the two, resulting in a permanent and predictable bond and creating a composite construction material. Reinforced Earth is flexible such that it is possible to build directly on compressible foundation soils or on unstable slopes.

The materials used to construct this wall consist of a cast-in-place leveling pad, standard precast concrete panels, galvanized steel reinforcement strips, permeable backfill, and filter fabric. The filter fabric is used on panel joints to prevent the backfill from seeping through the joints. The recommended backfill is sand, with less than 5-percent fines.

The length of the reinforcing strips depends upon the height of the wall. Generally, the strips are 0.7 times the height, but are not shorter than 8 ft, and they come in 2-ft increments. The required length of strips for this application is 12 ft. This base width is the same for the Armco Bin-Walls; therefore the excavation quantity and cost are the same for these two.

The construction of Reinforced Earth is said to be simple and repetitive. All premanufactured components are delivered to the jobsite. Using a crew of five and standard construction equipment, contractors can finish 750 to 1,000 sq ft of wall face per shift. After placing the cast-in-place leveling pad and the initial course of panels, the first lift of backfill is spread and compacted. The steel reinforcements are placed and bolted to the panels. A lift of backfill is spread and compacted over the reinforcing strips. This procedure is repeated until design height is reached.

The Corps of Engineers' estimate for the Reinforced Earth is approximately $1,500.00. The cost savings for this type over the Armco Bin-Walls and DoubleWall is in the price of the premanufactured components (precast panels and reinforcing strips).

TechWall

Traditional cantilever retaining walls can be an efficient and economical method of earth retention in areas where severe excavation or right-of-way restrictions prevent the use of such methods as Reinforced Earth. The use of counterforts in conjunction with retaining walls generally allows design efficiencies in the required quantity of concrete and reinforcing steel. However, counterfort retaining walls are economically difficult to cast at the project site. Therefore, the Reinforced Earth Company developed the TechWall product.

TechWall consists of reinforced precast counterfort wall sections and cast-in-place leveling pad and footings. The footing width is approximately 50 percent of the wall height. The leveling pad initially serves as a means of assuring vertical alignment and temporarily supporting the panels. These panels generally have weepholes incorporated into them for drainage purposes. Ultimately, the pad will be incorporated into the overall system as the shear key.

Since these walls use heavily reinforced counterforts, the precast concrete facing panels are relatively thin. The counterforts are tapered in shape from top to bottom. This tapered shape approximates the shape of the bending moment diagram produced by the horizontal earth pressures.

Once the leveling pad is placed, panels are set vertically with lifting bars placed through holes cast into the counterforts and are aligned with connecting plates that are attached between the two panels. After all of the panels are in place, a concrete footing is poured over the shear key to a level sufficient to cover the panel legs. When the footing is cured, backfill is placed and compacted. The required backfill is a pervious fill. Filter fabric lines the panels to aid in drainage and containment of the fill. The placement of the panels is based on conventional methods of tilt-up wall construction.

Although the excavation quantity is less than that for the Reinforced Earth—Mechanically Stabilized Backfill, the cost of the precast panels, delivered and erected, is higher. This difference causes the Corps of Engineers' estimate for the product TechWall to be approximately $100,000 more than that for the Mechanically Stabilized Backfill, which comes to a total of $1,600,000.

WaterLoffel

The WaterLoffel is a variant of the Loeffelstein, a spoon-shaped, stone-made product developed by Steiner Silidur A.G. of Switzerland in the mid-1970's. Like the Loeffelstein, the WaterLoffel modules interlock by wings or ears on each side. An additional cross-member creates two independent troughs that retain backfill. The units measure 25.6 in. deep, 26.5 in. wide, and 7.25 in. high. It is assembled in running bond without mortar, grout, or reinforcement.

All of the previous researched wall types were designed as a vertical wall. WaterLoffels are required to be constructed to a maximum slope of 70 deg and a minimum of 40 deg. This slope requirement allows the channel width to decrease from 55 ft to a 45-ft base and a 10-ft setback at the top of the wall. This narrower channel reduces the excavation quantity. WaterLoffel walls are constructed one unit deep. They are good with respect to expansion and contraction with changes in temperature since the modules are not mortared together. Directly behind and inside the modules is gravel.

Because of the slope requirement, a slightly different stability design was utilized. The results of this design showed that there was a global sliding problem and the wall was not stable even at the minimum slope of 40 deg. It was then decided that if sloping sides were

used instead of verticals, then riprap would be used. Because of this decision, a cost estimate was not done for this wall type.

Conclusion

The times are changing, and we must keep up with progressing technology and find new and hopefully better solutions to all of our problems, new and old. Each of these walls has its own benefits as well as drawbacks, but for the appropriate project would be very effective and beneficial. Currently, it appears that the Mechanically Stabilized Backfill manufactured by the Reinforced Earth Company will be used for construction of the channel wall. This alternative's cost is the least for this particular project on the River des Peres in St. Louis, MO.

References

Contech Construction Products, Inc. 1989. "Bin-Type Retaining Walls Type 2," Form BW2-101 Edition 1, Middletown, OH.

DoubleWall. 1982. "Interlocking Precast Retaining Wall System," Form 881, DoubleWall Corporation, Plainville, CT.

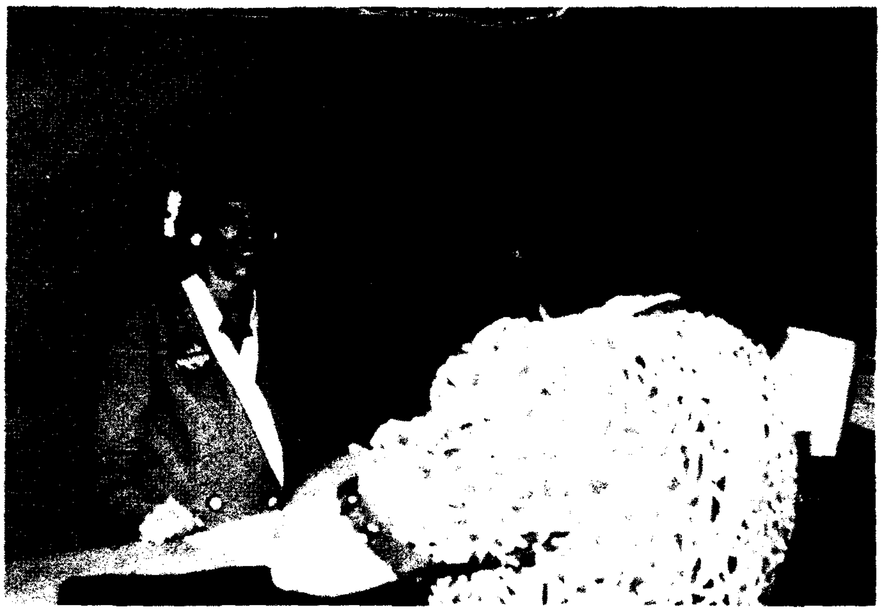
________. 1982. "Interlocking Precast Retaining Wall System," Form 983, DoubleWall Corporation, Plainville, CT.

________. 1982. "Interlocking Reinforced Concrete Retaining Wall Systems," Form 682, DoubleWall Corporation, Plainville, CT.

Reinforced Earth. 1988. "An Advanced Construction Technology," Reinforced Earth Company, Arlington, VA.

________. 1988. "Introducing TechWall, A Unique Precast Counterfort Retaining Wall System," Reinforced Earth Company, Arlington, VA.

Loffelstein. 1989. "Retaining Walls That Literally Come Alive," Seagren Industries, Inc., St. Louis, MO.



Structural Investigation and Repair
Cannon AFB Hospital

by

George E. Diewald, PE[1]

Abstract

An independent A/E firm was contracted by the Corps of Engineers to perform an investigation and a lateral load analysis of the existing hospital at Cannon AFB, New Mexico. The building was built in the mid-1960's. It is a three-story concrete frame with concrete pan joist floors and roof structures supported on a partial basement with short concrete pier columns in a crawl space. The lateral load analysis was performed to determine if the lateral load-carrying systems conform to the current wind and seismic codes. The structure is an "irregular" structure. This made the lateral analysis difficult to analyze and presented the A/E with some interesting problems. During the investigation, it was discovered that numerous short pier columns in the crawl space, under the first floor, had developed severe shear cracks. The A/E was given an additional contract to analyze these columns and prepare plans and specifications for their immediate repair.

Introduction

Cracking had developed in the floor slabs of the hospital at Cannon AFB, New Mexico. This hospital was constructed during 1965-1966. Beginning in 1987, an investigation was performed to determine the cause(s) for the floor cracking. This initial investigation led to the requirement for further studies and investigations into the structural integrity of this building. This paper will strive to describe the various investigations, studies, findings, and repairs which were performed on this structure over a period of 4 years from 1987 to 1991.

Building Description

The building is a three-story reinforced concrete structure consisting of one-way 14-in.-deep pan joist roof and floor slabs spanning in the north-south direction of the building with supporting beams in the east-west direction (grid lines 1-13, Figures 1, 2, & 3) supported by concrete columns on a 24-ft grid each direction. The exterior facade of the second and third stories consists of groups of three 4-ft double-tee precast wall panels separated by wide window units. The first story exterior facade consists of individual 4-ft-wide double-tee precast wall panels separated by narrow

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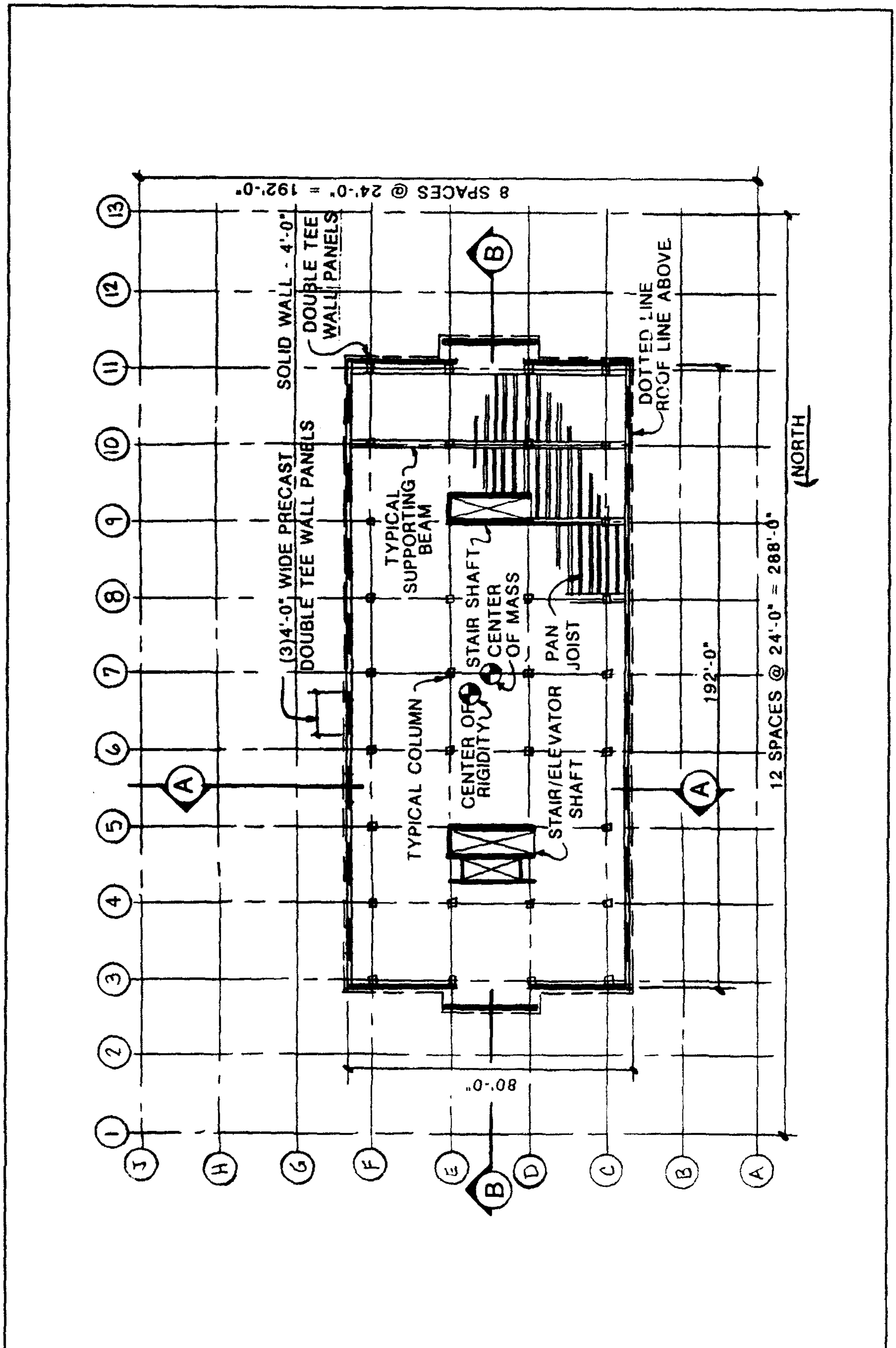

Figure 1. Third story and roof plans

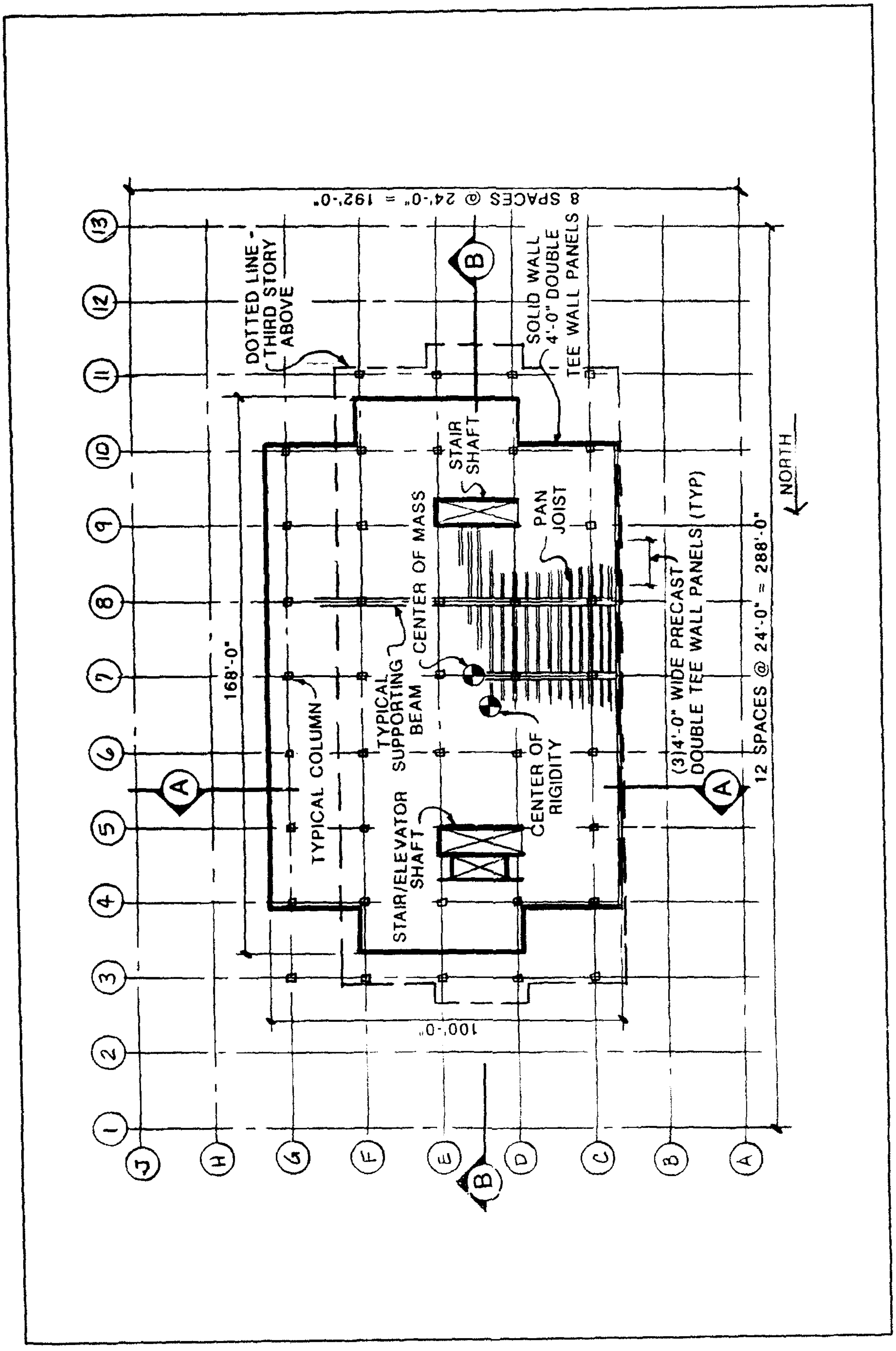

Figure 2. Second story plan

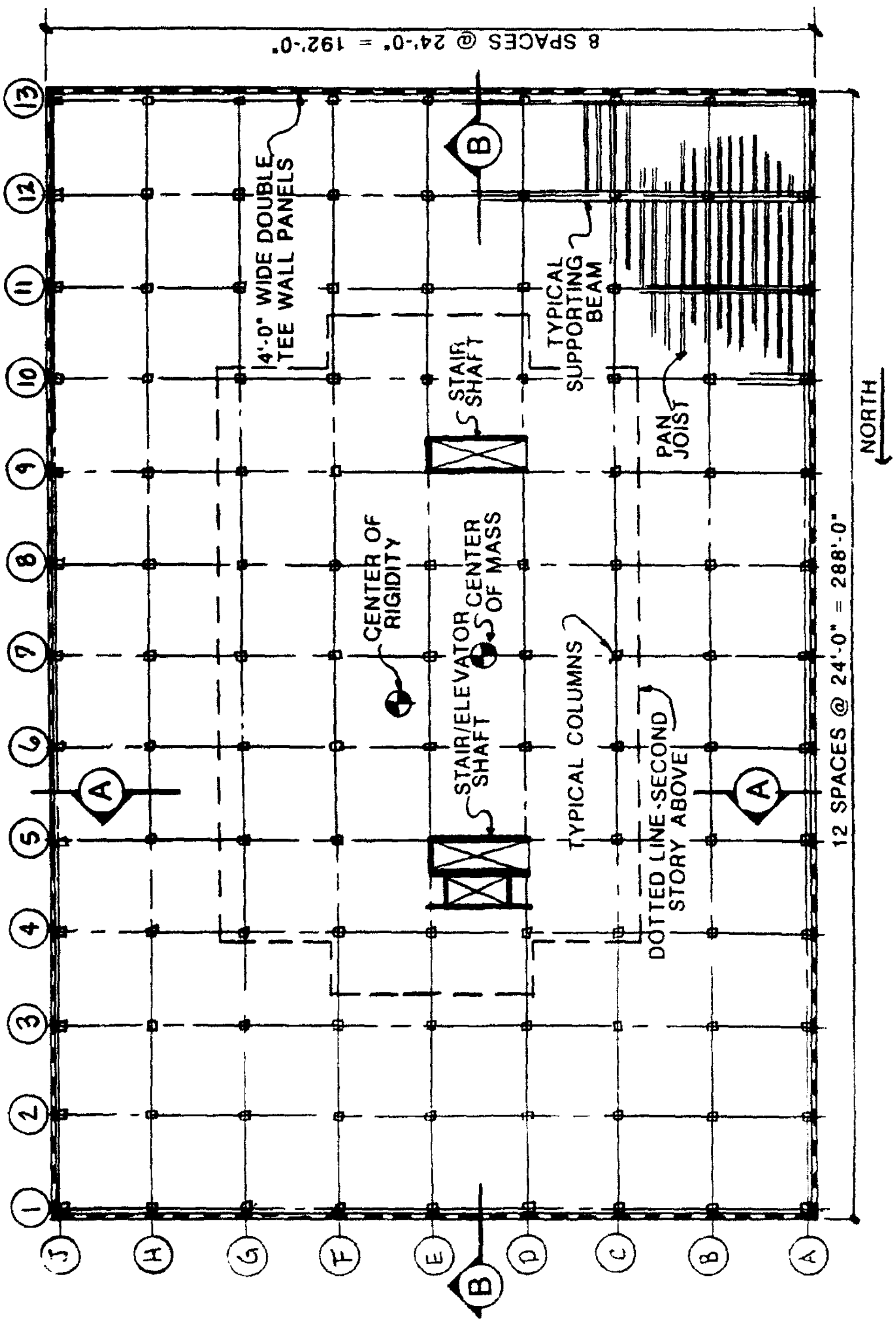

Figure 3. First story plan

window units. All of these panels are connected with embedded steel connections to the exterior beams of the floor and roof slabs.

The roof and the third story (192 ft N-S by 80 ft E-W by 12 ft high, Figures 1 and 5) overhang the second story in the north-south direction and are set back from the second story in the east-west direction creating a vertical irregular story over the second story. The second story (168 ft N-S by 100 ft E-W by 13 ft high, Figures 2 and 5) is set back from the first story in both directions. The first story (288 ft N-S by 192 ft E-W by 13 ft high, Figures 3 and 5) is the same configuration as the partial basement/crawl space level. At the partial basement/crawl space level (288 ft N-S by 192 ft E-W by 14-ft- high basement/2-ft to 4-ft-high crawl space, Figures 4 and 5) the north wall (grid line 1) consists of a combination of full-height walls at the basement, grade beams on short piers and full-height grade beams/columns. The south wall (grid line 13) consists of a combination of grade beams on short piers and full-height grade beams/columns. The east wall (grid line J) consists of grade beams on short piers. The west wall (grid line A) consists of full-height grade beams/columns. The partial basement walls within the crawl space function as a stiff element for lateral load distribution. Two elevator/stair shafts penetrate through the full height of the building to service the basement and the second and third stories from the first story.

March 1987 Survey and Analysis

Richard G. Vaughan & Assocs., Consulting Engineers, Albuquerque, NM, was contracted by the US Army Engineer District, Albuquerque, to investigate floor cracks in the loose-issue area (a heavily loaded floor area) of the hospital. This survey and subsequent analysis of the floor slab were performed to investigate the cause(s) of cracks which were visible in the top of the slab in the region of the north-south integral support beams.

Findings

It was discovered that the negative reinforcing of the pan joist, at the supporting beams, was missing and had not been installed as detailed on the original drawings. No positive moment cracking was observed.

Recommendations

It was recommended from this study that the entire hospital structure be investigated to determine if the negative reinforcing in all the pan joists had been left out during construction.

November 1987 Survey and Analysis

This survey, to determine if the negative reinforcing in all the pan joists had been left out during construction, and subsequent analysis were also performed by Richard G. Vaughan & Assocs. Negative moment cracking was evident in other areas of the hospital to varying degrees. The columns in the basement/crawl space area were observed during this survey and showed no signs of distress.

Findings

With the use of an R-meter and X-ray, it was determined that all the principal negative moment reinforcing was missing in the pan joists roof and floor slabs.

Recommendations

Because of this finding, it was recommended that a lateral analysis be performed on the structure to determine if the building complied with current seismic and wind codes. With the negative reinforcing missing in the pan joists, there was no longer an equivalent frame in the north-south direction of the structure.

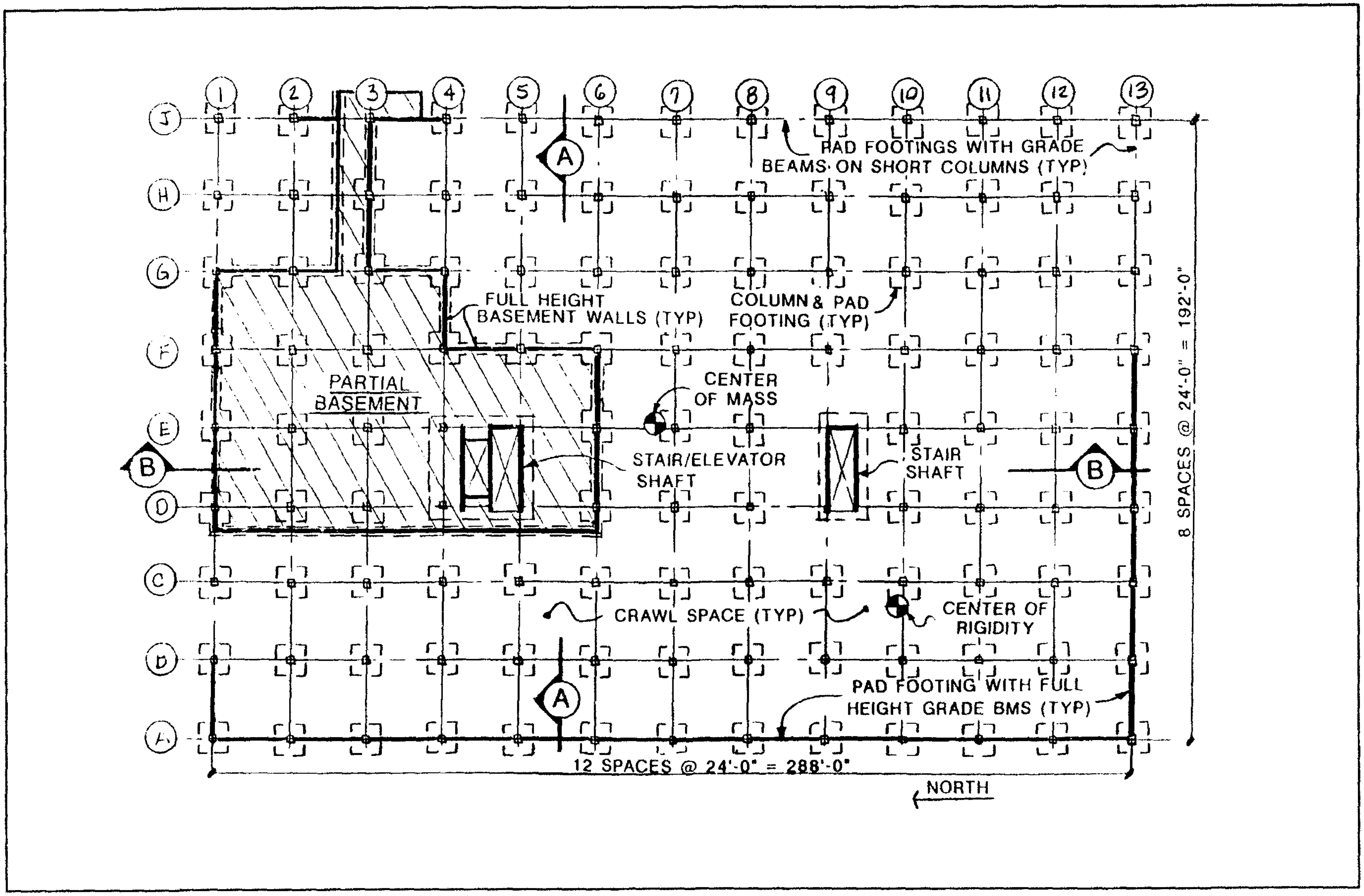

Figure 4. Basement/crawl space plans

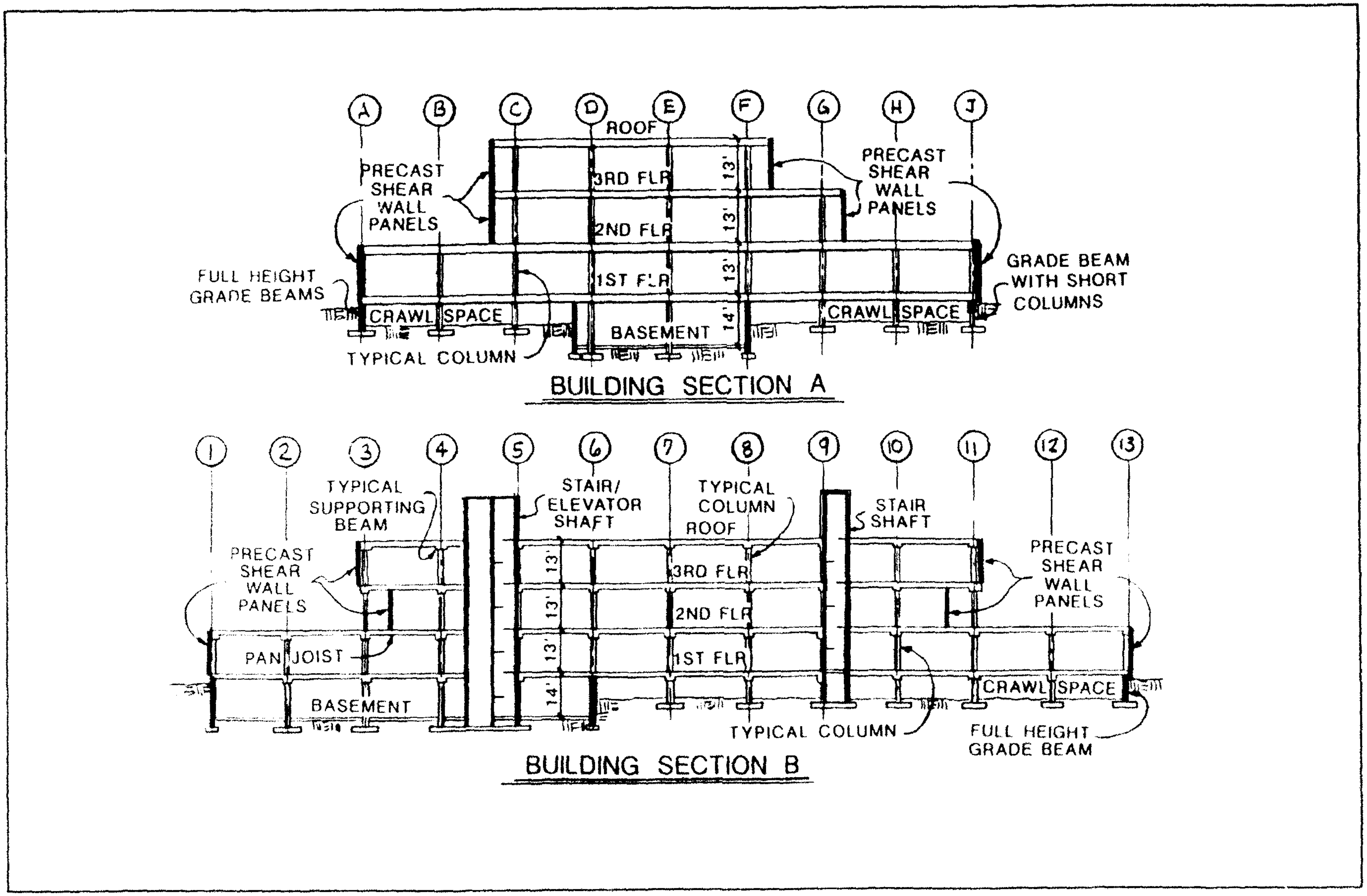

Figure 5. Building sections A and B

November 1989 Lateral Analysis

In November 1989 a lateral analysis study was performed by Webb-Leonard-Vaughan, Architect-Engineer, Albuquerque, NM. The purpose of this analysis was to analyze the structure for compliance to the current US Army Corps of Engineers (USACE) standards and Uniform Building Code seismic and wind loading requirements. This analysis was performed using the following assumptions:

- The exterior precast wall panels at the second and third stories are poorly connected to each other and act independently to each other. The shear capacity of the connections of the exterior wall panels to each other, along their vertical edges, makes a considerable difference in the lateral load-carrying capacity of the panels. If the panels act as one unit of many panels, they will resist considerably more lateral load than if they act as individual panels.

- The connection of the exterior precast wall panels to the roof and floor slabs has a safety factor of 3.0. The connection of the exterior precast wall panels to the supporting roof and floor slabs is critical, because it transfers the roof and floor diaphragm loads into the panels. This type of connection has to meet a Corps requirement for a safety factor of 3.0.

- The reinforcing dowels connecting the roof and floor slabs to the stair/elevator shafts were as specified on the as-built drawings. The capacity of the dowels which anchor the stair/elevator shafts to the roof and floor slabs is critical, because the dowels help to transfer the roof and floor diaphragm loads into the stair/elevator shafts.

- The strength of the roof and floor slab dowels to the stair/elevator shafts was 60,000 psi reinforcing. The strength of the dowel reinforcing will make a big difference in the amount of shear force which can be transferred into the stair/elevator shafts.

Results

Since the pan joist negative reinforcing was missing, the effectiveness of the pan joist equivalent frame in the north-south direction to resist any lateral loading was considered to be minimal. In the east-west direction of the structure, the analysis determined that the magnitude of the lateral load distribution to the concrete beam-column frames is small compared to the total lateral force on the structure. Therefore, results indicate a major portion of the lateral load is distributed to the exterior double-tee precast wall panels and the two stair/elevator shafts.

Recommendations

The A/E recommended another survey be conducted to substantiate the aforementioned four assumptions, since they are critical to the lateral stability analysis of the building.

April-May 1990 Survey

This survey was conducted to substantiate the assumptions made in the lateral analysis of November 1989. The following conditions existed for the four assumptions:

- The exterior precast wall panel vertical joints were connected by means of cast-in-place rebar anchors welded together with a common rebar. It was analyzed that these anchors were of minimal capacity, therefore the panels could not be considered to be acting together and had to be considered as separate units. This finding confirmed the assumption in the November 1989 analysis.

- The connections of the exterior precast wall panels to the respective roof and floor slabs were weakened by poor construction practices. The concrete around the anchor plates of the connection had been chipped away to allow for erection of the panels, and the anchor plates in the panels only partially line up with the anchor plates in the roof and floor slabs. The connections were analyzed to have an approximate

safety factor of 1.5, well below the required safety factor of 3.0. This finding greatly reduced the capacity of these anchor plates as analyzed in the November 1989 analysis.

- The dowels connecting the roof and floor slabs to the stair/elevator slabs are in place, but in several locations the connections were not as detailed on the as-built drawings. The dowels were spaced further apart and in some cases were a smaller size than specified on the as-built drawings. In some cases the safety factor is as low as 1.1 at the first story. This finding reduced the shear capacity between the roof and floor slabs and the shaft walls as analyzed in the November 1989 analysis.

- The dowel reinforcing strength tested to 60,000 psi. This finding confirmed the assumption in the November 1989 analysis.

With the determination of these assumptions, the A/E revised the lateral analysis that was performed in November 1989. During this survey it was observed that several columns in the basement/crawl space area showed signs of distress in the form of severe shear cracks in some and flexural cracks in others. These columns had shown no signs of distress in the November 1987 survey.

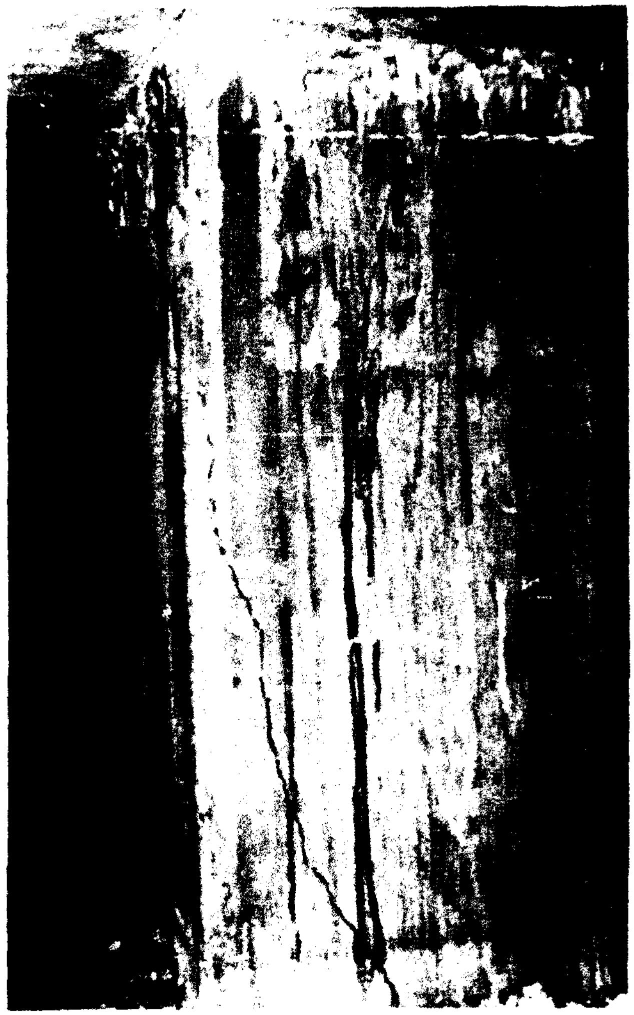

Figure 6. Shear crack in column H-11

Distressed Crawl Space Columns

Significant shear cracks were visible in columns H-3, H-6, H-7, H-10, H-11, G-11, F-12, G-12, and H-12. Column H-12 was the most severely cracked. The cracking pattern of the columns suggests the loading, which created the cracking, was bidirectional. Columns H-10, H-11, and H-12 yielded the reinforcing and had failed. The cracks were approximately 1/8 in. in width in these columns (Figure 6). All columns on line B had minor flexural cracks on the east faces. In addition, isolated minor cracking was observed in the columr around the column-beam connections on lines A and J. It was reported by maintenance personnel at the hospital that on May 30, 1988, a very severe storm occurred in the area. This storm produced 4 in. of rain in 2 hours, extensive flooding, and very high winds estimated to be in the range of 50 to 70 MPH. One tornado funnel cloud was also reported in the vicinity of Cannon AFB. Maintenance personnel reported that the suspended ceiling system in offices in the southeast corner of the building fell during this storm. These offices are in the vicinity of the most severely distress columns H-10, H-11, and H-12. It is therefore theorized that the hospital was hit by tornadic winds, causing vibrations in parts of the hospital, and displacements substantially in excess of the column shear capacities.

Discussion of the Lateral Analysis

The Cannon AFB Hospital, as originally constructed, is not in compliance with current USACE standards and the 1988 Uniform Building Code lateral seismic requirements. The system, a dual system, does not comply with the requirements that the vertical space frame be capable of withstanding 25 percent of the base shear. Because of the weight of this building, seismic action controlled over wind for the lateral analysis.

The first, second, and third stories are for the most part symmetrical, with both the center of mass, and the center of rigidity near the center of the building (Figures 3, 2, & 1, respectively). The exterior precast wall panels are relatively stiff elements and carry a majority of the lateral load. The exterior precast shear wall panels of the third and second stories do not occur on the column center lines. The lateral loads of the third and second stories have to be transferred from the offset vertical exterior precast shear wall panels into the lower floor diaphragm, through the slab diaphragms, past column frame(s), and into the offsetting vertical exterior precast shear wall panels of the story below. This type of load path transfer is termed vertically irregular.

The second story is set back approximately two column lines (56 ft) in the north-south direction and one and two-thirds column lines (40 ft) in the east-west direction (Figure 3). The second-story precast shear wall panels transfer lateral load into the roof of the first floor. The lateral load has to be transferred through the first-story roof diaphragm, past two column lines (1 & 2 or 11 & 12) in the north-south direction and one column line (B or H) in the east-west direction, and into the vertical precast shear wall panels on the exterior of the first story and finally down to the basement/crawl space area.

As described previously in the "Building Description," the basement/crawl space lateral supporting structure is very irregular with very stiff full-height basement walls in the partial basement, has a combination of shallow grade beams on short columns, and very stiff full-height grade beams/columns in the crawl space. The center of mass and the center of rigidity in the basement/crawl space area are not coincident (Figure 4). Therefore, in the distribution of the lateral loads, considerable torsional loading is exerted on the structure.

The first lateral analysis performed in November 1989 assumed that the loads in the basement/crawl space area were resisted entirely by the full-height basement walls. With the discovery of the cracked columns in the crawl space, this thought process was reevaluated and the stiffness of the all elements in the area were taken into account. The most severely distressed columns were on column line H, next to exterior column line J. The lateral resisting elements on line J are shallow grade beams on short columns, which are the weakest of the basement exterior wall resisting elements. This could account for the worst distressed columns being on line H, since the columns on line H would be forced to contribute to the resistance of the loads induced into line J. The short crawl-space columns are 48 in. high and contain a large amount of vertical reinforcing (12- by 12-in. columns, 7.06 percent, and 18- by 18-in. columns, 3.9 percent). Therefore, they are controlled by a brittle shear mode of failure, which is very undesirable, rather than a ductile flexural failure. It is estimated that the shear forces exerted to the columns by the lateral displacements were likely as high as three-fourths of the ultimate capacity of the columns in shear. The safety factor of these columns is very difficult to estimate, but based on research data, these columns could have safety factors as low as 1.3. Such a low safety factor is unacceptable for main vertical load-carrying members in structures. This information was obtained from research testing on very similar columns, as published in the *ACI Structural Journal* (1988 and 1989).

Columns on line B exhibited flexural cracking. This is due in part to the fact they are longer columns of approximately 76 in. This flexural cracking is on the east face usually about 12 in. below the top of the column.

The above referenced research indicates that both the strength and ductility are reduced substantially with successive applications of load on this type of column, and that with increased cycles of loading subsequent shear cracking could develop.

Conclusion and Repairs

The A/E concluded the April-May 1990 report with a closing paragraph that best describes the lateral load-carrying system of the building: "When all the irregularities of the building are considered, normal redundancies and reserve ductile capacities of the structure as a whole are nonexistent. Current standards for reinforced concrete buildings, because of recent failures under lateral loads, are emphasizing detailing of joints, connections, and other critical elements to provide for increased reserve energy absorbing capability of the structure as a whole. This building currently has little or no reserve capacity."

Immediate Repairs

The basement/crawl space columns will require immediate repair and strengthening to restore their loss of shear capacities. It was recommended that the building be evacuated or the columns be repaired as soon as possible. It was decided to repair the columns. The A/E was contracted by the Corps to design and prepare a set of construction documents for these repairs. The construction of the column repairs is currently in progress. The repair method consists of encasing the existing columns in a 6-in. reinforced concrete collar. The collar concrete will be 12,000-psi slurry grout.

Future Repairs

The repair of the basement/crawl space columns will not remedy the lateral deficiencies, but will repair only the distressed columns. Additional remedial measures must be taken to upgrade the lateral resisting elements before the building structure will comply with current wind and seismic standards and codes. These additional remedial measures will include but not necessarily be limited to following items:

- The installation of additional strategically located concrete shear walls within the first story and in the basement/crawl space area. This would help reduce the torsional effects in the basement/crawl space and the vertical irregularities of the building.
- The repair of the exterior precast wall panel connections to the roof and floor slabs.
- The installation of additional exterior precast shear wall panels between the existing panels at all levels of the building.

These remedial measures are scheduled for design in FY-93.

References

American Concrete Institute. 1988 (Sep-Oct). "Lateral Load Response of Strengthened and Repaired Reinforced Concrete Columns," *ACI Structural Journal*, Detroit, MI.

________. 1989 (Jan-Feb). "Response of Reinforced Concrete Columns to Simulated Seismic Loading," *ACI Structural Journal*, Detroit, MI.

Richard G. Vaughan & Associates, Consulting Engineers. 1987 (Mar). "Review and Analysis of the Loose Issue Storage Area Floor Cracking," Albuquerque, NM.

________. 1987 (Nov). "Field Review of the Pan Joist Floor System Throughout the Hospital," Albuquerque, NM.

Webb-Leonard-Vaughan, Architect-Engineer. 1989 (Nov). Preliminary Lateral Load Analysis of the Hospital," Albuquerque, NM.

________. 1990 (Apr-May). "Field Review and Testing to Verify/Not Verify Whether Components of the Structural System Were in Place to Resist Lateral Loads," Albuquerque, NM.

Webb-Leonard-Vaughan, Architect-Engineer. 1990 (May). "Final Report on the Structural Condition of the Hospital," Albuquerque, NM.

_______. 1990 (Nov). "Lateral Load Analysis for Shear Walls - Recommendations," Albuquerque, NM.

Rehabilitation of Buildings 330, 331, and 338 Holloman AFB, New Mexico

by
Ted M. Solano[1]

Abstract

Buildings 330, 331, and 338 are two- and three-story dormitories, constructed in the 1950's. The buildings were renovated as part of a continuing base improvement program. The existing structural systems consist of moment-resisting concrete space frames with masonry infill between frame bents. The majority of the existing masonry infill was removed and replaced with a metal stud wall system. The floor and roof diaphragms are concrete slabs which were placed integral with the frames. The foundation system consists of spot footings which support the frame columns. Grade beams which support exterior walls are present, spanning between perimeter columns. The structural additions to the buildings consist of a balcony corridor system around the entire building for exterior access, upgrade of lateral load-resistant system since the existing system did not meet current code requirements, and replacement of existing foundation grade beams which were severely deteriorated. The new balcony beam-column system (utilizing precast panels, concrete masonry unit (CMU) walls, and concrete columns) and the existing building frame equally support the balcony corridor and roof extension. New CMU pier shear walls and or steel bracing resist 100 percent of the wind and seismic lateral loads.

Introduction

Buildings 330, 331, and 338 are unaccompanied, enlisted, personnel housing (UEPH) dormitories located at Holloman AFB, near Alamogordo, NM. The three buildings are of similar construction and were rehabilitated as part of a base improvement program. Buildings 330 and 331 are identical, two-story structures which were constructed in the early 1950's. Rehabilitation construction for both of these buildings was under the same contract. Construction began in June 1989 and was completed in August 1990. Construction cost for both buildings was approximately 3.1 million dollars. Building 338 is a three-story structure which was constructed in the late 1950's. Rehabilitation construction began in March 1990 and was completed in February 1991. Building 338 is one of two buildings being rehabilitated under the same construction contract. The other building is currently under construction. Construction cost for both buildings is approximately 4.4 million dollars.

Existing Building Conditions

The original architectural layout for all three buildings was similar. Access into individual sleeping quarters was through an interior,

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central corridor which ran in the longitudinal direction of the building. An exterior stairway, for access into the second- and third-floor corridors, was located at transverse building ends. At Buildings 330 and 331, two large bathroom facilities were located centrally on each floor and served all dorm occupants for that floor. At Building 338, every two rooms were provided with a common bathroom facility. The original second-floor plan, a demolition plan, for Building 338 is shown in Figure 1.

The building structural systems for all three dormitories are similar. The vertical and lateral load resistance systems consisted of moment-resisting space frames with masonry infill between frame bents. The space-frames at Buildings 330 and 331 consist of concrete columns and beams. Roof and floor diaphragms are concrete slabs which are supported by and were placed integral with the frame beams. At Building 338, the space frame system at the outer "wing" portion of the building consists of concrete columns and a flat slab system placed integral with the columns at diaphragm levels. Concrete beams placed integral with the flat slab system are present at the building perimeter. The space-frame system at the central "core" portion of the building consists of concrete columns and beams. Roof and floor diaphragms are concrete slabs which are supported by and placed integral with the concrete beams.

The foundation systems for all three buildings consist of a slab on grade with spot spread footings which support isolated columns. Grade beams, which support the exterior CMU walls, span between perimeter columns.

Evaluation of Existing Building Structures

Extensive concrete deterioration was present at the perimeter grade beams at the foundations of all three buildings. Grade-beam reinforcement was exposed and corroding at several locations. Longitudinal rebar and stirrup locations were evident by crack patterns on the exterior faces of several grade beams (Figure 2). Grade-beam crack patterns corresponded with reinforcement locations (moment steel and stirrup) on as-built drawings. At Building 338, the exterior concrete overhangs at the diaphragm levels were also badly deteriorated (Figure 3). At all buildings, it was noticed that the grade-beam concrete above grade had deteriorated more than that below grade. Also, concrete deterioration at the grade beams and at the overhangs was greater on the sunny sides of the buildings.

In an effort to determine the overall integrity of the existing concrete at the building structures, cores were taken at random locations throughout the entire building space frame (Figure 4). Random core locations consisted of foundation grade beams (above and below grade), exterior and interior concrete columns and beams, and also interior concrete floor slabs. Interior cores were taken at all floor levels. Core locations within the buildings were limited to areas that were not private quarters (i.e., hallways, stairways, lounge, etc.), since the buildings were occupied at the time the cores were taken. Exterior core locations were limited to those areas that the coring machine could physically be lifted and secured to. Special equipment was not present to raise the core machine above this level. Compressive strength tests were conducted on all cores taken and tests determining the density and general composition of the concrete were conducted on selected pieces of a few random cores.

Results of Evaluation

Drawings indicate that the design concrete compressive strength for all the buildings was 2,500 psi. Compressive strength tests for all cores were well above this value except at the exterior grade beams at Building 338. The average concrete strength at these grade beams (Building 338) was 2,360 psi for above-grade cores and 4,313 psi for below-grade cores. Concrete density tests conducted on the core samples revealed that the densities were normal and did not have a wide range, indicating that the concrete was homogeneous. Concrete composition tests indicated that the concrete was nonair-entrained and that the cement

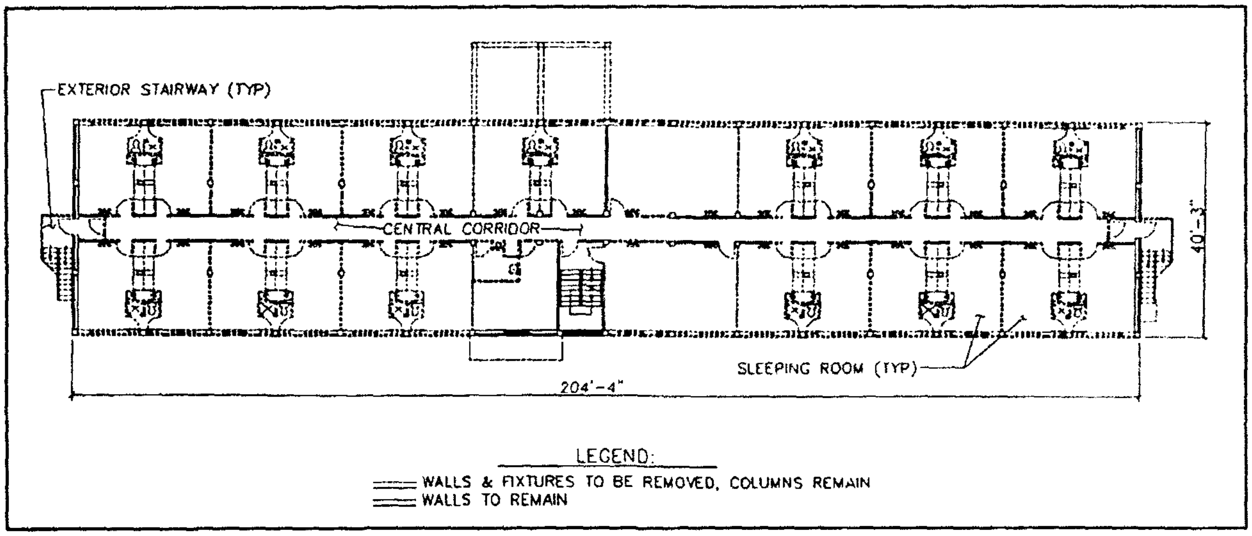

Figure 1. Building 338: Second floor demolition plan

Figure 2. Building 338: Grade beam crack patterns corresponding with reinforcement locations

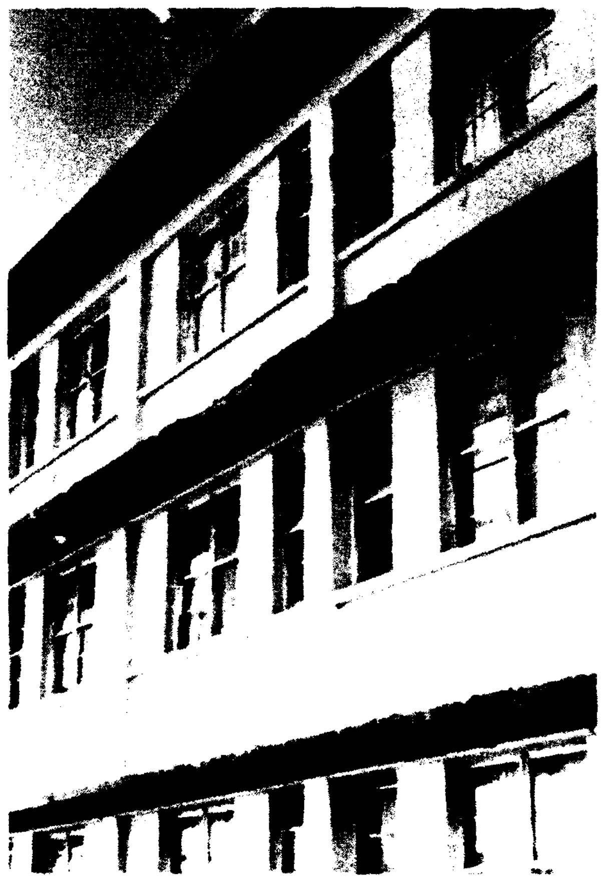

Figure 3. Building 338: Concrete deterioration at building overhangs

Figure 4. Building 330: Core drilling at an interior concrete beam

content and aggregate properties were not unusual. Corrosive soils are present at the site; however, tests revealed no form of chemical attack in the concrete.

A high groundwater table is present in the vicinity of the buildings. As a result, the grade beams can become saturated. Being critically saturated, coupled with freeze-thaw cycles will cause concrete to deteriorate. These conditions are believed to have caused the grade-beam concrete deterioration present at the buildings. Corrosion of the reinforcing steel was caused by the increased permeability of the concrete due to microcracking, resulting from the freeze-thaw process. The reinforcement corrosion process causes localized expansion, which in turn caused the crack patterns on the faces of the grade beams (Figure 2). The overhang deterioration at Building 338 also resulted from the freeze-thaw process. The water source in this case was most likely ponding rainwater and snow. The freeze-thaw theory is further supported by evidence of more severe concrete deterioration at those locations where freeze-thaw cycles occur more frequently (i.e., above grade versus below grade and on the sunny side of the buildings).

It was determined that the grade beams at all buildings had deteriorated to such an extent that they were not functioning as originally designed. As a result, grade beams at all buildings would be removed and replaced. The concrete overhangs at Building 338 were scheduled to be removed in the rehabilitation construction and resulted in no additional work or cost.

Rehabilitation Construction

New architectural layout

The new architectural layout was similar for the all three buildings. A new balcony corridor system around the entire building perimeter was added for exterior room access. The new architectural second floor plan for Building 338 is shown in Figure 5. New bathrooms were located within the original central corridor. Every two rooms now shared a common bathroom facility. Most of the existing masonry infill between the concrete frames was

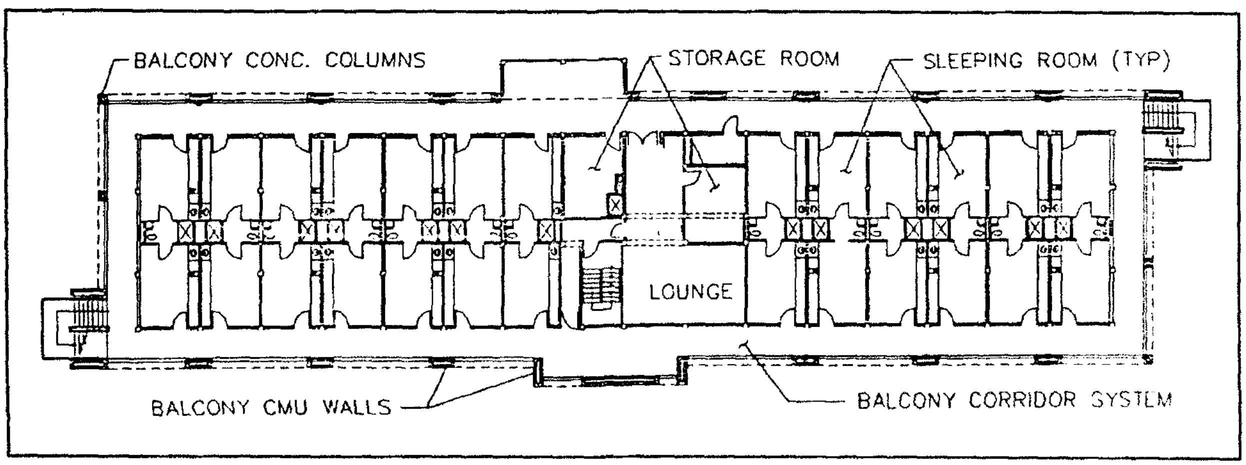

Figure 5. Building 338: New architectural 2nd floor plan

replaced with stud walls due to the extent of new wall penetrations (resulting from floor plan changes). All exterior and interior building surfaces were refinished. Existing plumbing, HVAC, and electrical systems were removed or abandoned, and new systems were supplied. Essentially, only the building space frame system would remain after demolition (Figure 10).

New balcony corridor system

The new balcony corridor system is 6 ft wide at floor levels and is covered at the roof level. A new beam-column system and the existing building frame equally support the balcony corridors at each floor level and the roof extension. The new balcony beam-column system consists of concrete columns or CMU walls which support precast concrete panels. The panels support the new roof or balcony corridor addition equally with the existing building frame. Figure 6 is a typical section through the balcony corridor system at a precast panel in Building 338.

New lateral load-resistant system

A new lateral load-resistant system was provided at each of the buildings. Existing space frames did not meet the current seismic requirements for concrete moment-resisting space frames (type B) as specified in TM 5-809-10, "Seismic Design for Buildings." The new lateral load- resistant systems at each building were designed to resist 100 percent of the lateral design load. The existing building frames would resist vertical loads only.

The following new lateral load-resistant systems were provided at Buildings 330 and 331. In the transverse building direction, braced frames were provided, consisting of steel cross bracing between existing concrete frames. In the longitudinal building direction, a combination of braced frames and CMU shear walls was provided.

At Building 338, br ced frames were provided in the transverse building direction. The braced frames consisted of steel cross bracing between existing concrete frames. Due to architectural considerations, a braced frame system was unacceptable in the longitudinal building direction. CMU shear walls were used to resist design lateral loads in this direction.

At all buildings, the CMU walls within the new balcony corridor systems were utilized as the shear walls in the new lateral load-resistant systems. The shear walls are 2 ft wide and range in length from approximately 8 to 16 ft. Braced bays at all braced frame locations within the buildings extended from the foundation to the roof diaphragm. The lateral load-resistant system at Building 338 is shown in Figure 7. A typical braced bay between diaphragm levels is shown in Figure 8.

Due to the rigid diaphragms present at all of the buildings, lateral loads are distributed to resisting elements (i.e., braced frames or shear walls) based on their relative rigidities. Since braced frames and shear walls were used in the longitudinal building direction at Buildings 330 and 331, the relative rigidities

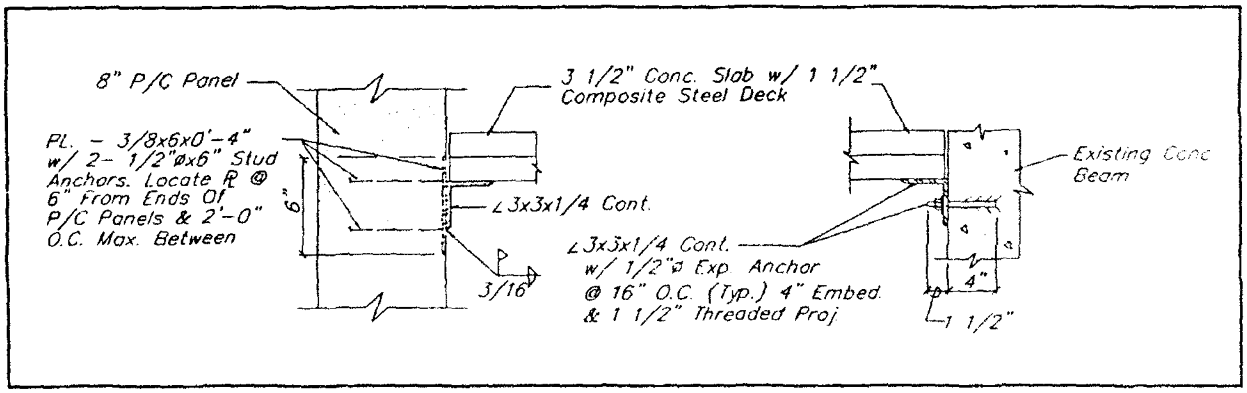

Figure 6. Building 338: Section through balcony corridor system at precast panel

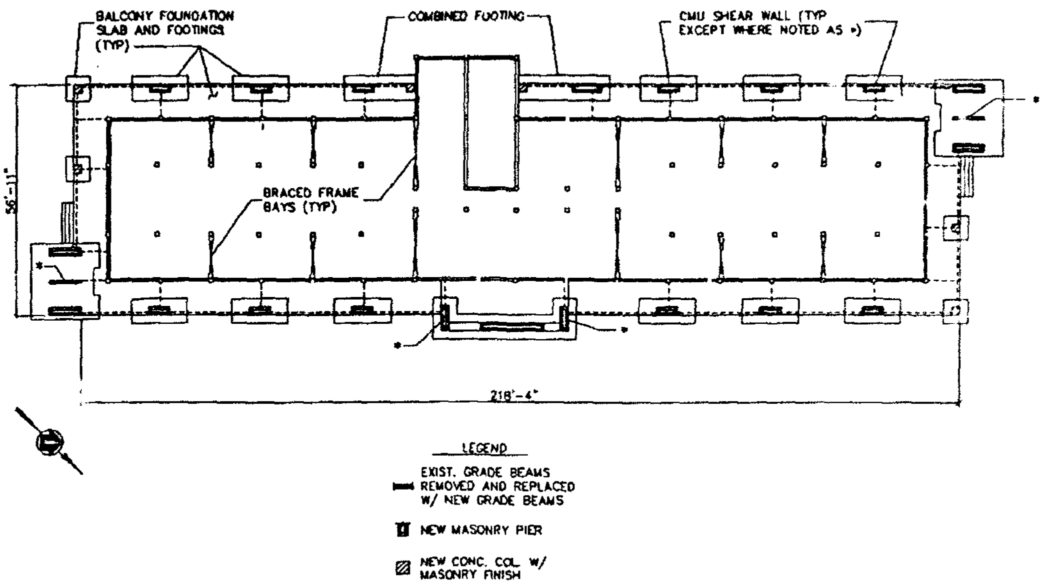

Figure 7. Building 338: New foundation plan

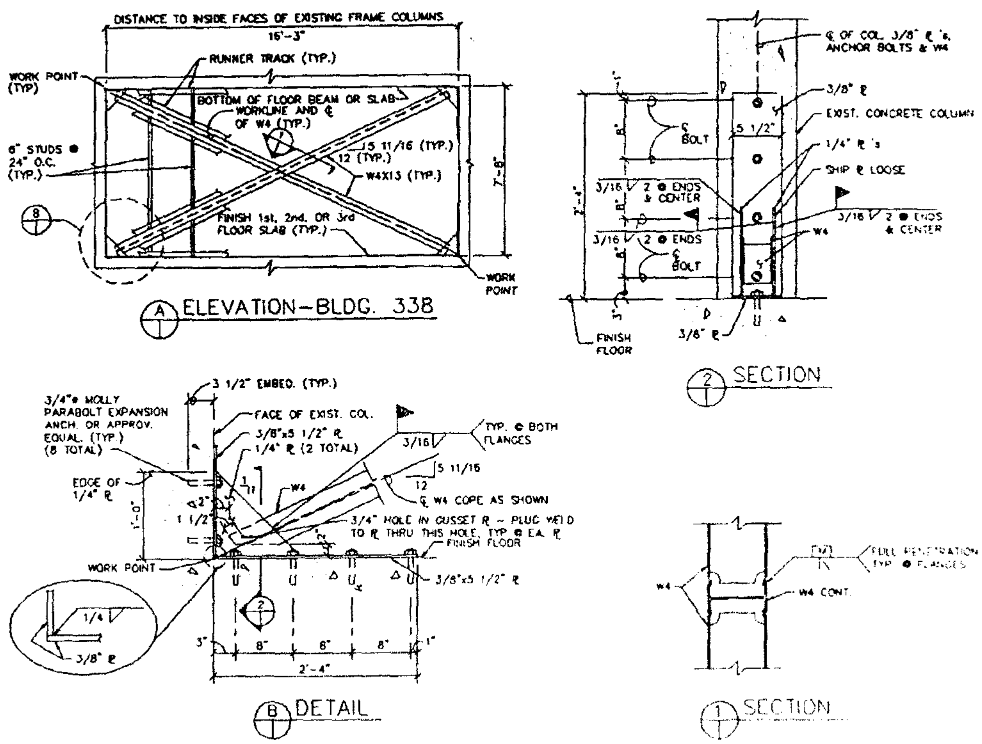

Figure 8. Building 338: Braced bay

of the braced frames with respect to the masonry shear walls had to be determined to calculate the torsional force (associated with the rigid diaphragm) to resisting elements. The rigidities of the shear walls were determined by applying a 1-kip load to each shear wall and using the resulting deflections to calculate the rigidities. Figure 6-11a in TM 5-809-10 was used to determine the deflections. Proper adjustments were made to account for actual material, wall thickness, and load (1 kip) used. The rigidities of the braced frames were determined by modeling the frames on a computer and applying a 1-kip load. Resulting deflections from the computer output were used to determine the rigidities.

Lateral loads are transferred to the new balcony shear walls which are exterior to the original building concrete diaphragm, via a new rigid diaphragm (i.e., the new balcony corridor diaphragm system). A concrete slab with a composite metal deck was used for the new corridor diaphragms. The new diaphragms were connected to the existing building diaphragm and to the balcony masonry walls or precast panels as required to transfer the design lateral loads to the new shear walls. The precast panels served as struts, transferring the lateral loads to the shear walls. Seismic loading conditions governed overwind conditions at all buildings. Total design lateral loads at Building 338 were greater than those at Buildings 330 and 331, since an additional story is present at this structure. As a result, additional balcony shear walls and braced frames with more capacity were required to resist the design lateral load at Building 338.

Foundation design and construction

The grade beams at all buildings were removed and replaced due to the extensive grade beam deterioration present (Figure 9). Preventative measures were taken to avoid future concrete deterioration problems associated with the freeze-thaw process and the corrosive soils present at the building sites. All new concrete, 3 ft above grade and below, had a compressive strength of 4,000 psi, was type V, had an air content by volume of approximately 5.5 percent, and contained no pozzolan. A waterproofing surface treatment consisting of two coats of linseed oil was applied to all new and existing concrete exposed to the earth or elements.

Figure 9. Building 331: Grade beam removal

The footings below the new balcony masonry shear walls were designed to resist their applicable lateral loads. As a result, footings that were larger and thicker were required to transfer the lateral load to the foundation soil (Figure 7). Existing footings at braced frame locations were checked to verify that design lateral loads could safely be transferred to foundation soils.

At Building 338, an existing mechanical and laundry room extended from the main portion of the building. The building extension is one story and was incorporated into the new balcony system. New concrete columns, which help support the balcony corridors and roof, were placed adjacent to the existing building extension. Since each column abutted the existing building, the new footing edge adjacent to the existing foundation below is flush with the face of the column. A spot footing could not be used below these columns due to the high bearing pressures resulting from footing eccentricities. As a result, combined footings were used at these locations. Each combined footing supports the column adjacent to the existing building extension and the masonry shear wall nearest to the column (Figure 7).

Closing Statements

Design conditions for the buildings were not ideal as the as-builts were of poor quality, and existing conditions could not be verified due to building occupation at the time of design. Few construction problems were encountered, however. The steel bracing connections to the existing concrete frames had to be modified at Buildings 330 and 331 due to existing conditions that differed from those shown on the as-builts. Asbestos was also encountered at Buildings 330 and 331. This slowed the construction process, since removal was not included in the original contract. The building transformation that took place was quite impressive as Figures 10 and 11 indicate.

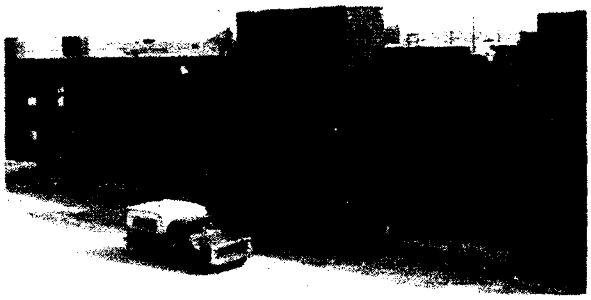

Figure 10. Building 330: Building skeleton after demolition

Figure 11. Building 331: Finishing stages of construction

AFLC Child Development Centers Cold-Formed Framing

by
Eric Fry[1]

Abstract

The US Army Engineer District, Louisville, has been tasked to design six child development centers for AFLC. The first three projects—Wright Patterson AFB, Ohio, Tinker AFB, Oklahoma, and McClellan AFB, California—are new structures. The other three projects—Robins AFB, Georgia, Kelly AFB, Texas, and Hill AFB, Utah—are proposed add-ons to existing child care facilities. The design schedule for these facilities has been set at a very accelerated pace. Each structure will vary in size and square footage. However, each structure will consist of one (or more) main rectangles which will be 70 feet (21.3 m) wide and vary in length. The standard 70-foot (21.3 m) width is being accomplished by using standard modules for the individual rooms: toddler, pretoddler, infant, etc. Construction shall consist of 8-inch (20.3 cm) masonry exterior walls with a 4-inch (10.2 cm) brick facade supported on continuous concrete footings. In addition, two interior 8-inch (20.3 m) masonry hallway walls will run the length of the structure. The structure shall have a 4 on 12 gabled roof across the 70-foot (21.3 m) width. The final decision for the framing was to use cold-formed steel rafter and joist assemblies supporting a standing seam roof. This system provides many desired qualities and a few difficulties, which will be summarized in this report.

Introduction

Of the six child development centers (CDC) to be designed for the Air Force Logistic Command (AFLC), only one has been designed to final at the time of this writing—Wright Patterson Air Force Base (WPAFB), Ohio. Therefore, the WPAFB-CDC will be the main topic of discussion throughout this report. WPAFB is located in South Central Ohio, and is the sight of the AFLC Headquarters. Its existing child care facility houses fewer than 100 children and is inadequate overall. The new CDC will be a 12,000-square foot (1115 square meters) facility capable of caring for 125 children. The project includes multi-purpose rooms for different age groups, isolation and storage rooms, kitchen area, additional playground area, and fire protection and support facilities/utilities. The accelerated design process consisted of basic design, progressing to 20 percent in just two weeks. This was accomplished by the initial "Charette" in which all heads met to perform concept level design in only two days. The design then proceeded to a 60 percent onboard review, 60-90 percent, and 90 percent to final. The building shape consists of a main rectangle approximately 70 feet ((21.3 m) by 125 feet (38.1 m) with two rectangular wings on each end, one approximately 38 feet (11.6 m) by 38 feet (11.6 m), the other approximately 50 feet (15.2 m) by 32 feet (9.75 m)). See

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[1] Structural Engineer, US Army Engineer District, Louisville; Louisville, KY.

Figure 1 for the basic building plan. The building construction consists of a standing seam metal roof supported by the cold-formed framing system. The cold-formed framing system is made up of C-channel roof rafters and ceiling joists bearing on exterior masonry walls and interior masonry corridor walls in the main rectangle. The rafters are on a 4 on 12 pitch creating an open attic space. The masonry walls are supported by continuous reinforced concrete foundation walls and footings. The lateral force resisting system consists of cold-formed strip bracing welded to the bottom of the ceiling joists, transferring the load to the shear walls below. The end shear walls are masonry along with the interior corridor shear walls. Metal stud walls serve as the interior shear walls transferring lateral loads to the foundation by diagonal strip bracing welded to the studs. The framing maintains a 1-inch (2.54 cm) expansion joint between the main rectangle and each end wing. The floor slab will be a floating 4-inch (10.2 cm) thick concrete slab-on-grade reinforced with welded wire mesh for temperature and shrinkage requirements. The floor slab will have crack control joints and will be supported by a 6-inch (15.2 cm) capillary water barrier with a membrane vapor barrier. The somewhat unique use of light gauge cold-formed steel C-Channels for the attic and roof framing led to some distinct advantages and a few difficulties.

Advantages of Cold-Formed Steel Framing

Open attic

By using light gauge framing, with joists and rafters screwed together back to back at 2 feet 0 inches (0.61 m) on center (O.C.), a 10 inch (25.4 cm) deep CEE-section was needed. An open web steel bar joist roof system spaced at 5 feet-0 inch (1.52 m) O.C. would have resulted in twice the joist depth, thus inhibiting the open attic space needed for mechanical and HVAC. The 4 on 12 roof pitch resulted in a gable roof with a ridge height clearance of approximately 11 feet-6 inches (3.50 m). The open attic space was a much desired quality for both installation and maintenance of mechanical and electrical equipment.

Lightweight framing

The 10 inch (25.4 cm) deep CEE-sections used were spaced at 2 feet-0 inch (0.61 m) O.C., very similar to conventional wood framing, which is often spaced at 1 foot-4 inch (0.41 m) O.C. However, the joists and rafters are lighter per foot and stronger per pound than wood. That is, the lightweight steel has a higher strength to weight ratio resulting in economical and efficient design and easier handling in the field or plant. With the spacing of

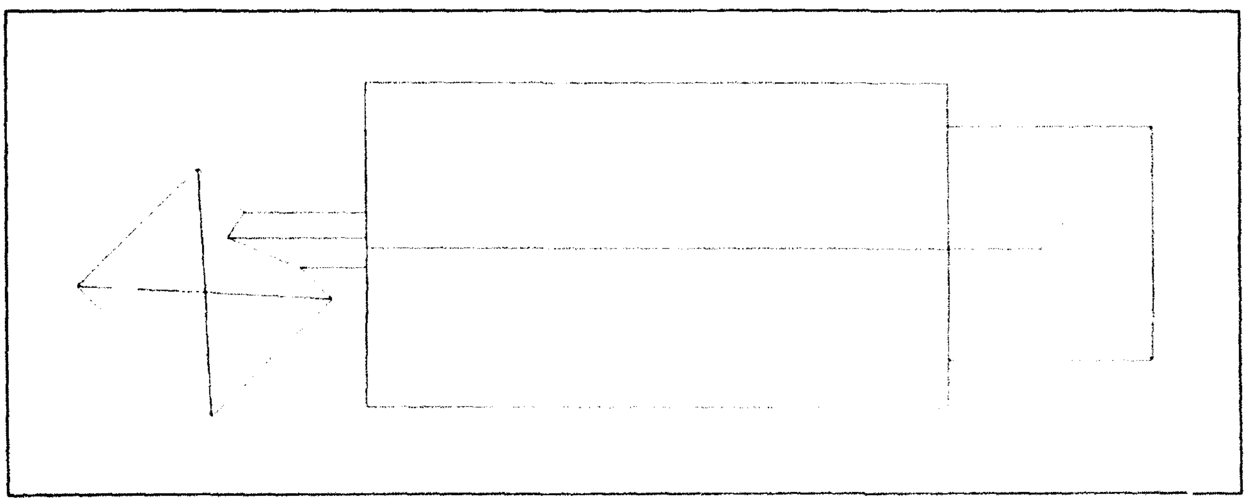


Figure 1. Roof plan WPAFB CDC

the steel, and its availability in very long lengths, the result is fewer lineal feet of material, and fewer joists are needed and handled. As mentioned above, a steel bar joist roof system may have been competitive as far as overall weight is concerned, but the advantage of the open attic would have been lost.

Strength and quality of steel

Aside from being lighter and stronger, the steel won't warp, split, check, shrink, or rot. Many of the maintenance costs so commonly associated with wood joists are either eliminated or reduced substantially. Other costly nuisances caused by sagging, shrinking, and twisting of framing are not a problem with dependable light gauge steel framing. Also, CEE-sections are normally hot-dipped galvanized steel for maximum durability and protection.

Fast installation

Light gauge steel CEE-sections are installed using conventional tools and methods. Connections are commonly made with self-drilling screws, but heavier gauge sections may also be welded or bolted. Also, members are available prepunched to provide access through for wires and pipes. Joists are cut to exact lengths at the fabricator's plant, and are ready to install the moment they arrive at the job site in winter or summer. Many developers have stated that steel joists go up faster, saving several manhours per building unit, without the considerable waste involved.

Repetitive design and details

The framing plan used for the WPAFB CDC consisted of joist and rafter systems spaced at 2 feet-0 inch (0.61 m) O.C. These systems (with the open attic) are close to being preassembled trusses. The majority of these frames are typical gable shapes, as shown in Figure 2. Others are partial gable shapes (interrupted by skylight wells) as shown in Figure 3. The use of such repetitive frames, with simple connections, over the major portion of the building results in both efficient design and construction costs.

Non-combustible

Light gauge CEE-sections won't support combustion. The WPAFB CDC structure requires a type IIN construction in accordance with the Uniform Building Code. This type means that the structural members are left unprotected (non-fireproofed), and these steel members will not readily support combustion.

Difficulties with Cold-Formed Steel Framing

Connections

As with any framing system, difficulties will arise. In the case of this Child Care Center, connections for the most part were standard, and could be easily made with conventional tools and methods. However, this roof system also contained several dormer type structures and a gabled "cathedral ceiling" type structure over the vestibule and lobby areas. Hence, the connection details between roof rafters become more complex. A well trained contractor will still be very capable of carrying out such connections, but the situation does require a stringent attention to detail on the part of the structural engineer.

Cold bridging

Another difficulty associated with roof framing connections is the problem of cold bridging. This happens when a metal roof member is exposed to the cold air, when it is actually desired to have it protected with a vapor barrier and insulation. The result is cold air contact and condensation on the member. The original construction plan of this CDC was to have the insulation and vapor barrier down at the attic level. However, because of a HVAC need, the insulation was raised up to the rafter level. This led to two difficulties. First, an insulating strip was now required between the connection of the standing seam metal roof to the room framing. Secondly, another potential

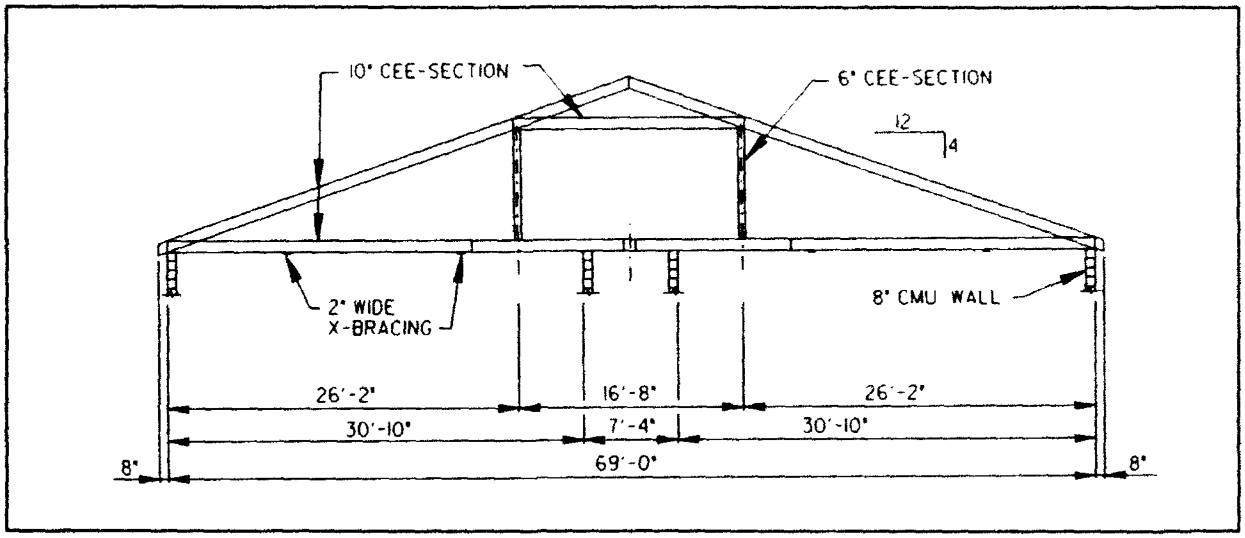


Figure 2. Typical roof framing

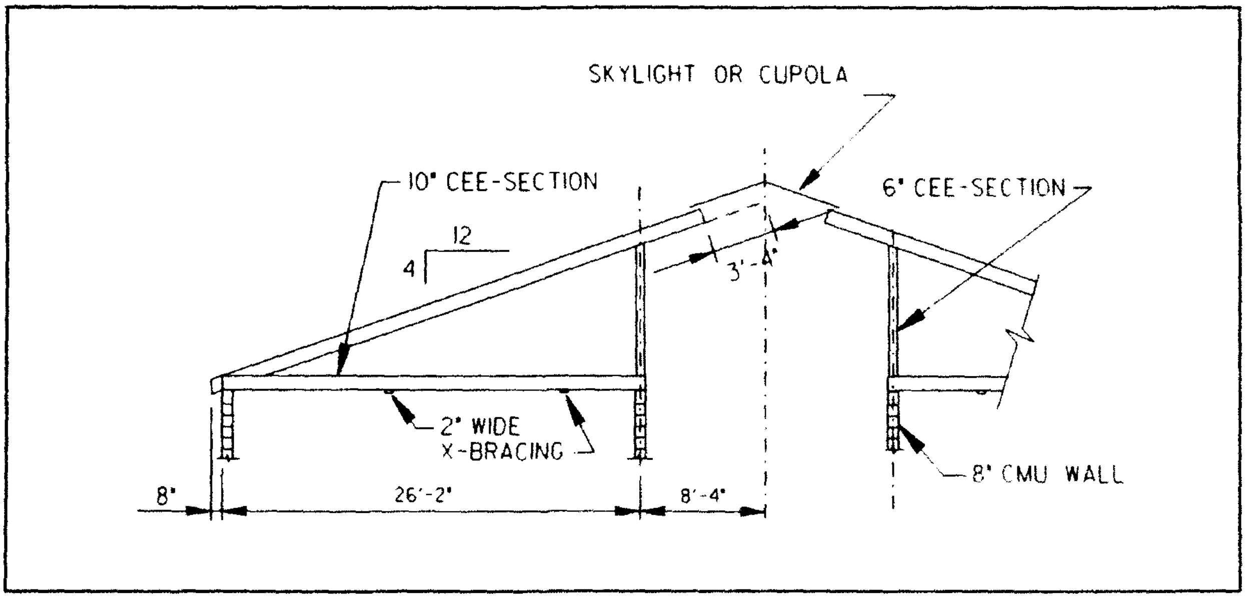


Figure 3. Roof framing near skylights

cold bridge was occurring at the connection of the rafter to the attic joist. As a result, a plywood gusset plate was added to the connection to break the cold bridge.

Conclusions

As long as a proper attention to details is carried out, a cold-formed lightweight steel framing system proves to be a competitive roof framing system. Many contractors today have already made a successful use of cold-formed stud framing systems. But, the use of the light gauge metal for roof framing is also proving to be a viable alternative. As a result, it is believed that a very efficient and constructible solution has been determined for the structural system for the Wright Patterson Air Force Base Child Development Center and the other AFLC CDC's to come.

References

International Conference of Building Officials. 1988 (May). "Uniform Building Code," Whittier, CA.

US Army Corps of Engineers. 1990 (Aug). Memorandum, "Scope of Work Discussions," FY 90 Child Development Center, Wright-Patterson AFB, OH.

US Army Corps of Engineers. 1990 (Nov). "Final Design Analysis, AFLC Child Development Center," Wright-Patterson AFB, OH.

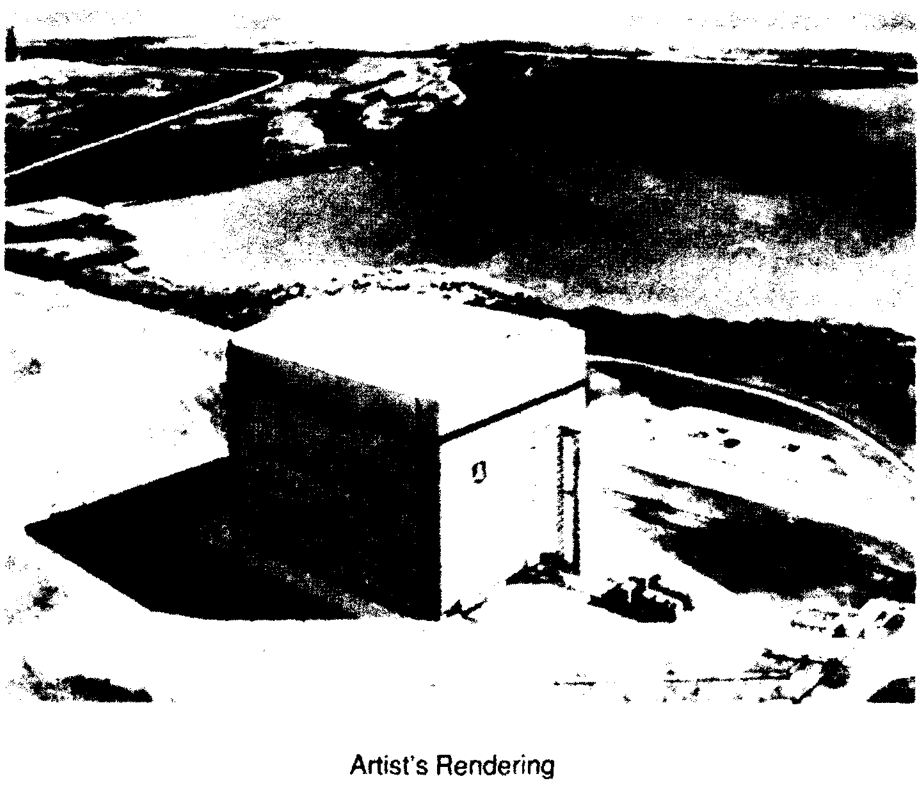
United States Steel. 1976 (Mar). "Super-C Steel Joists," Pittsburgh, PA.

The Solid Rocket Motor Assembly Building (SMAB)

by
Khim N. Vira, PE[1]

Abstract

The Solid Motor Assembly Building (SMAB) will provide a facility to receive, assemble, checkout, integrate, and prepare the upgraded Solid Rocket Motors for the Titan IV Missile Systems. This facility will allow the three segment, 400-ton Solid Rocket Motor to be stacked to its f: ll 114-foot height and to be placed on a transporter for movement to the launch pad on a specially prepared rail system. The main building is a 59,200-square foot, 250-foot high steel frame structure supported by a pile foundation and enclosed by metal siding and metal roof. The other supporting facilities are a guard house, a wastewater treatment facility, storm water detention, paving, other site and utility work, and a security system. The total cost of this project is about $40 million. The project is 50 percent complete as of 30 April 1991.



Artist's Rendering

[1] Structural Engineer, US Army Engineer District, Mobile; Mobile, AL.

Introduction

The Solid Rocket Motor Assembly Building (SMAB) is an Air Force project located at Cape Canaveral Air Force Station, Florida. The Cape Canaveral AFS is adjacent to NASA's Kennedy Space Center located on the central east coast of Florida on the Atlantic Ocean. These are the Nation's primary facilities for space launches including space shuttles and other Department of Defense launches. The Titan IV Rocket is an alternate to the space shuttle to launch DoD satelites and other non-commercial satellites.

The SMAB is a 59,200-square foot metal building. The SMAB is 320 feet long by 185 feet wide by 250 feet high. The building will be used to receive, inspect, and assemble Titan IV Solid Rocket Motor Vehicles. An assembled core vehicle comprised of a first and second stage will be moved to the SMAB from the Vertical Integration Building. The stacked Solid Rocket Motor Units (SRMUS) will be attached to the core vehicle in the SMAB. The completed assembly will then be moved by transporter to the launch pad for the addition of the upper stage, payload fairing, and space vehicle.

The purpose of the SMAB is to minimize the assembly time on the launch pad by maximizing the amount of assembly performed off the launch pad, thus providing an increased launch rate. The SRMUS segments will be received in the horizontal position on a heavy duty rail car and lifted by a 220-ton bridge crane. The segments will then be assembled on the vehicle transporter by a 500-ton and 200-ton bridge crane.

Major Design Features

Building structure

The building is a steel frame structure supported by a pile foundation. The building was designed for a hurricane force wind of 100 miles per hour. The building has 54 foot wide by 185 foot high, six-leaf vertical lift doors for moving the transporter from the building to the launch pad with a complete Titan IV Rocket vehicle in the vertical position.

3-D model study

Due to the size and complexity of the SMAB, a 3-D model was developed to analyze the space frame structure. The program chosen to do this analysis was the STARDYNE MICRO 32-BIT. The model included all the structural members with the exception of the columns and girders which support the 220-ton crane, the girts, and those roof purlins which are not in the panel points of the roof trusses. The siding and roofing are modeled as plate elements which did not participate in the stiffness of the model but transfered their dead load, as well as the wind pressures, into the model. These plate elements transformed these uniform loads into concentrated forces acting on the nodes of the model.

Three types of space frames were considered at the start of the analysis. These were a rectangular pattern of support columns with K bracing for the truss members, a rectangular pattern of support columns with cross bracing for the truss members, and a triangular pattern of support columns with tubular cross bracing providing both horizontal and vertical bracing.

It was determined that the rectangular pattern with cross bracing would be the easiest to erect and give the most useable space for the facilities, platforms, and walkways located around the perimeter of the building.

Foundation

The foundation for the structure consists of a pile mat. There are approximately thirteen hundred (1300) piles spaced approximately 6 feet-6 inches apart with extra piles under the columns and other areas of high concentrated load. The thickness of the mat is 5 feet-0 inches at the outer perimeter and 3 feet-6 inches in the open area. Due to the cost of the foundation, an elaborate test pile procedure was established to optimize the pile type, size, and spacing. The piles used were 14 inches square

prestressed concrete 75 feet long with a 70-ton capacity.

Wind loads

The building is located a few thousand feet from the Atlantic Ocean. The wind load used for this facility was 100 mph. The wind load design meets Air Force Manual AFM 88-3, Chapter 1, and ANSI A58.1. The wind exposure factor used was D. In order to reduce very high wind forces, the corners of the building were rounded off. Also the girts and purlins were doubled at the corners and perimeter of the building to withstand high wind forces.

There was some controversy about the wind loads concerning whether or not a 33 percent overstress in the steel members could be used when W+DL governed the design. ANSI 58.1-82 is vague on this, however, as AFM 88-3 does permit the overstress, it was decided to allow the 33 percent overstress. The first interior bay, approximately 25 feet wide, around the perimeter of the building is designed to resist the wind loads.

The roofing and its fasteners were designed to withstand an uplift of 144 pounds per square foot at all four corners of the building (approximately 25-foot by 25-foot areas). The remaining roof area was designed to withstand an uplift of 80 pounds per square foot. All roof areas were designed to withstand a downward pressure of 80 pounds per square foot.

The siding and its fasteners were designed to withstand a suction of 180 pounds per square foot at all four corners of the building. The remaining wall areas were designed to withstand a suction of 82 pounds per square foot. All wall areas were designed to withstand a pressure of 82 pounds per square foot.

All structural members, roofing, and siding were permitted a 33 percent increase in allowable stresses due to wind loading only. However, fasteners were not permitted the 33 percent increase in allowable stresses.

Structural steel painting

Structural steel surfaces were shop painted by one coat of inorganic zinc applied to a 3-mill thickness. This also served as a finish coat for the steel. Only minor touchups were required in the field. This paint system saved considerable money for this project and was satisfactory to the user.

Acknowledgement

This paper relied on the Design Documents prepared by Bechtel National, Inc., San Francisco, CA.

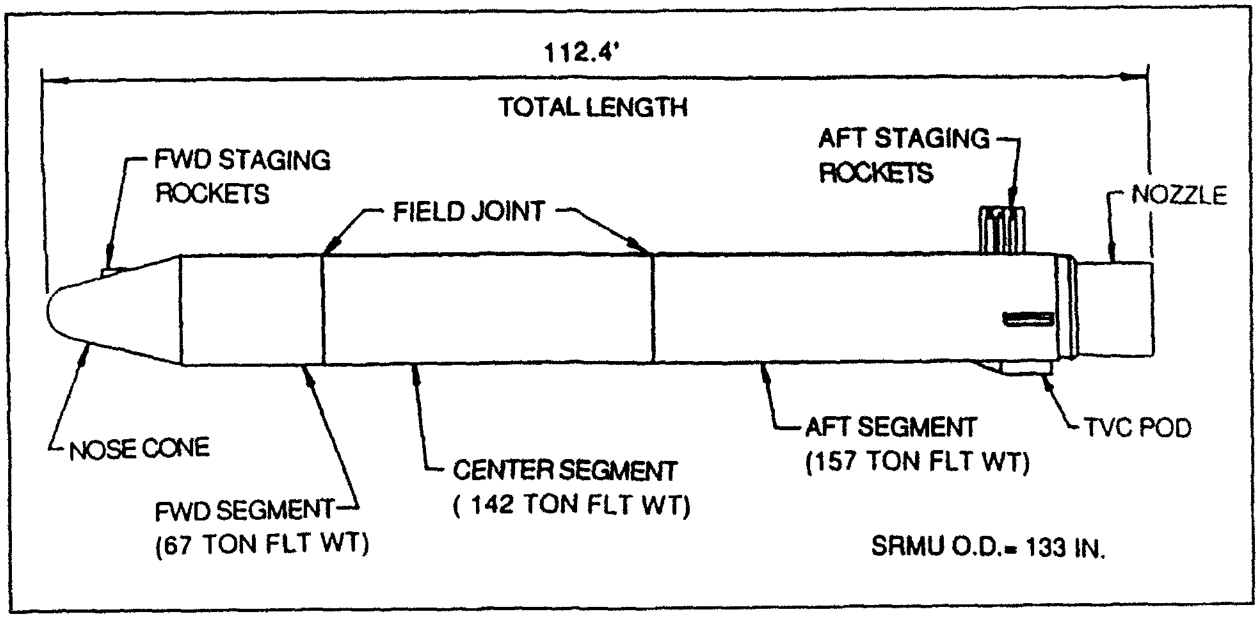


Figure 1. SMAB/SRMU, SRMU outboard profile and weights

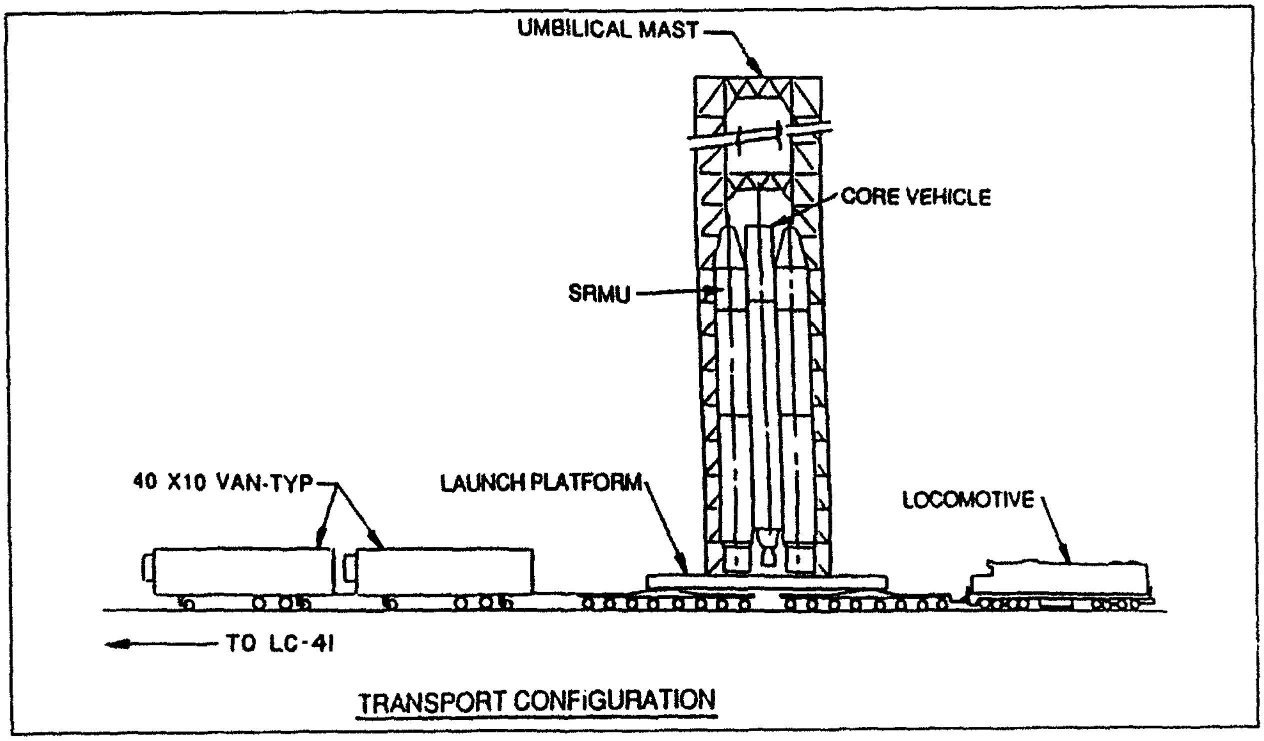


Figure 2. TIV full stack to LC-41

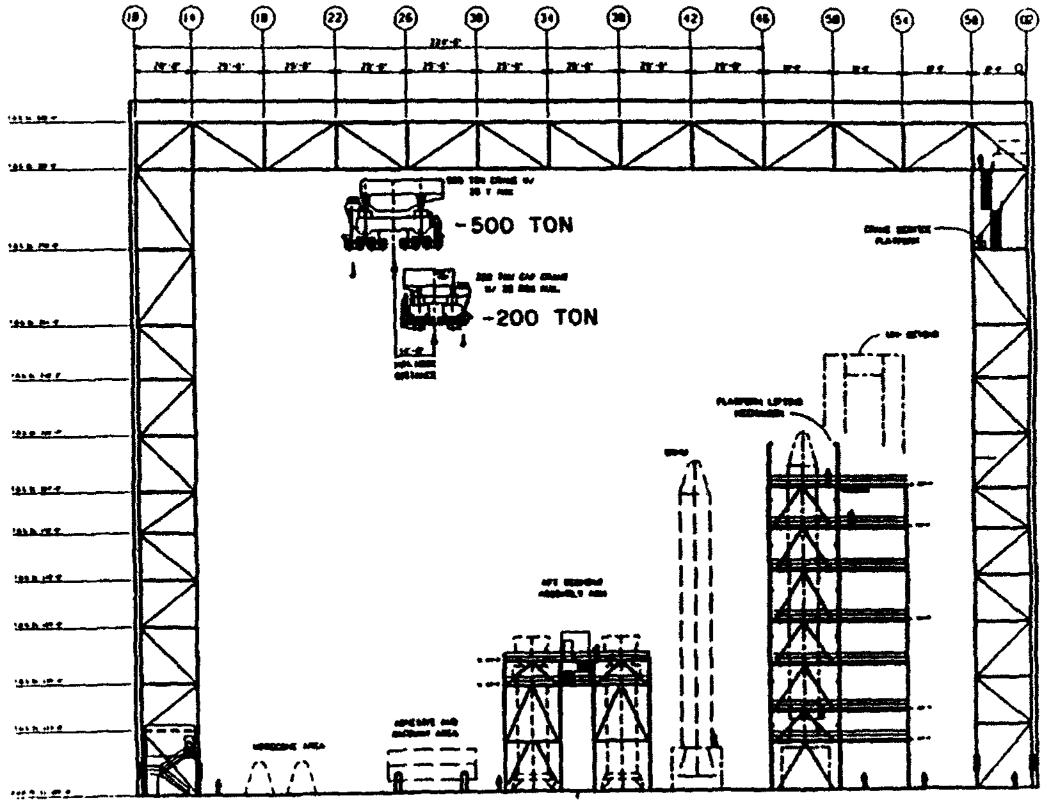


Figure 3. Longitudinal section

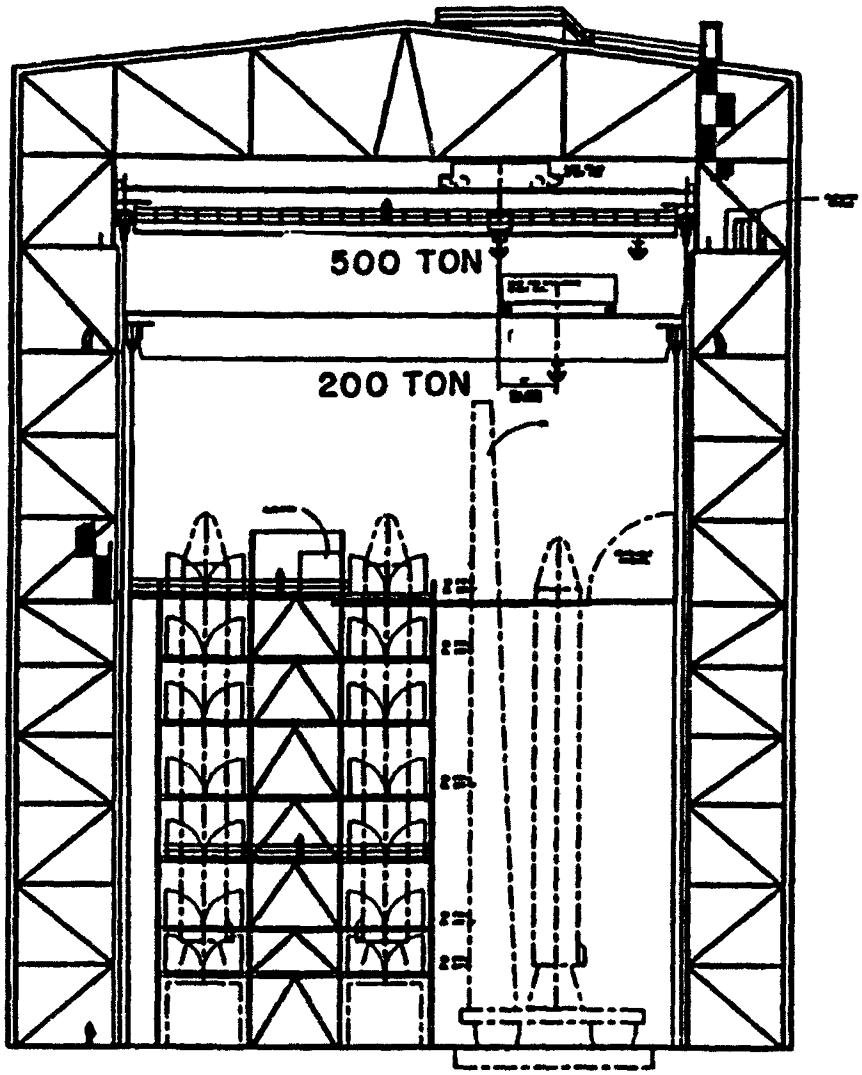


Figure 4. Cross section

Figure 5. Pile layout

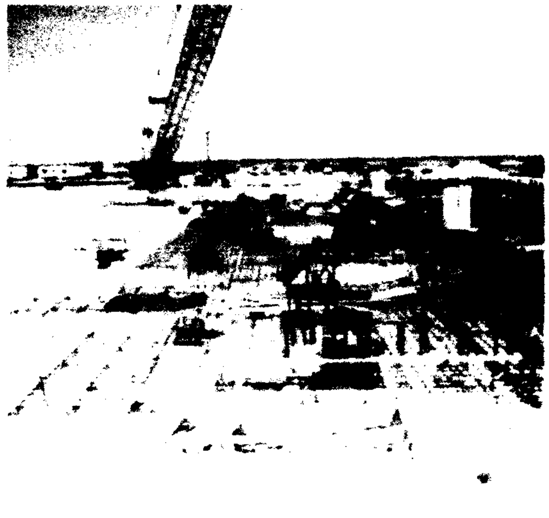


Figure 6. Mat reinforcement

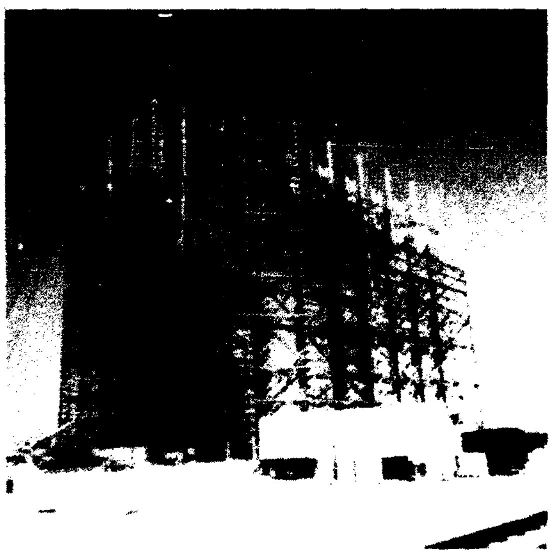


Figure 7. Steel erection

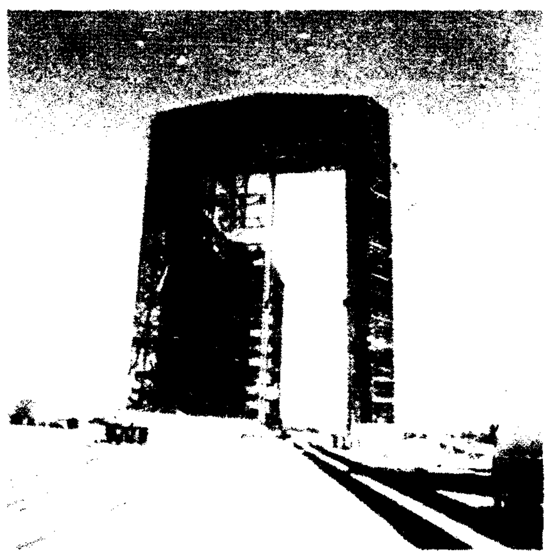


Figure 8. Steel framing—south elevation

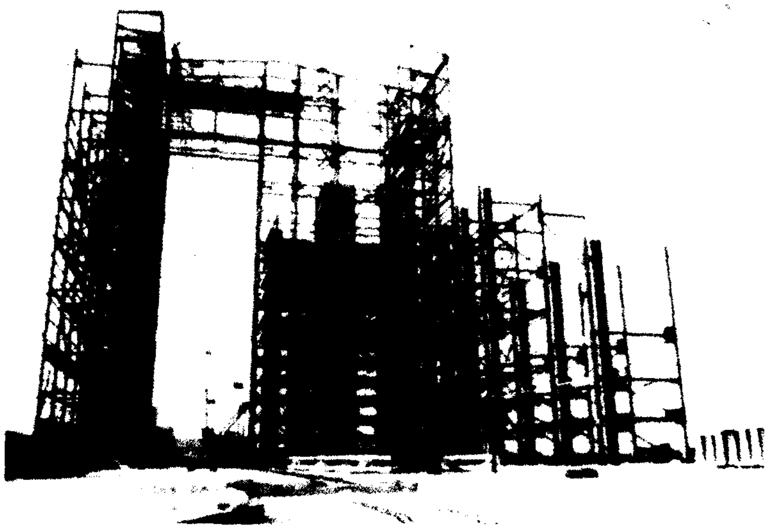


Figure 9. Steel framing —north elevation

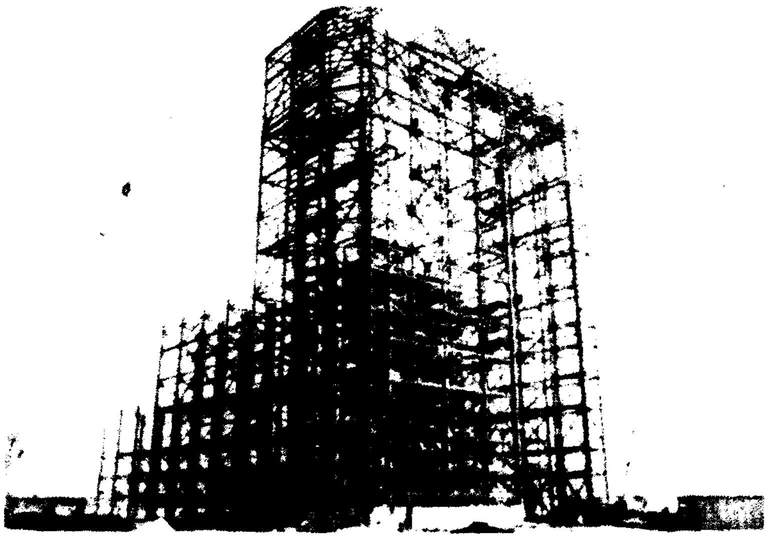


Figure 10. Steel erection —northeast corner

Unit Chapel
Fort Campbell, Kentucky

by
Jeffrey E. Bayers[1]

Abstract

The Unit Chapel project at Fort Campbell, Kentucky, utilized a standard definitive design in its development. The chapel consists of a vaulted main worship area surrounded by one story functional areas and circulation space. Material choices for the worship area structure included structural steel and exposed structural glued-laminated wood, the latter being chosen by user's preference. The wood framing was designed by the laminated wood manufacturer. The low rise areas were designed as load bearing masonry walls with steel joists and metal deck roof system. The main element of concern in the design was the separation of the two structural systems to accommodate movement between them. In the areas where they joined, coordination of deflection limits were further complicated due to the desire for exposed wood framing, "clerestory" windows to introduce natural light into the worship area, and moveable partitions used in separating the main rooms.

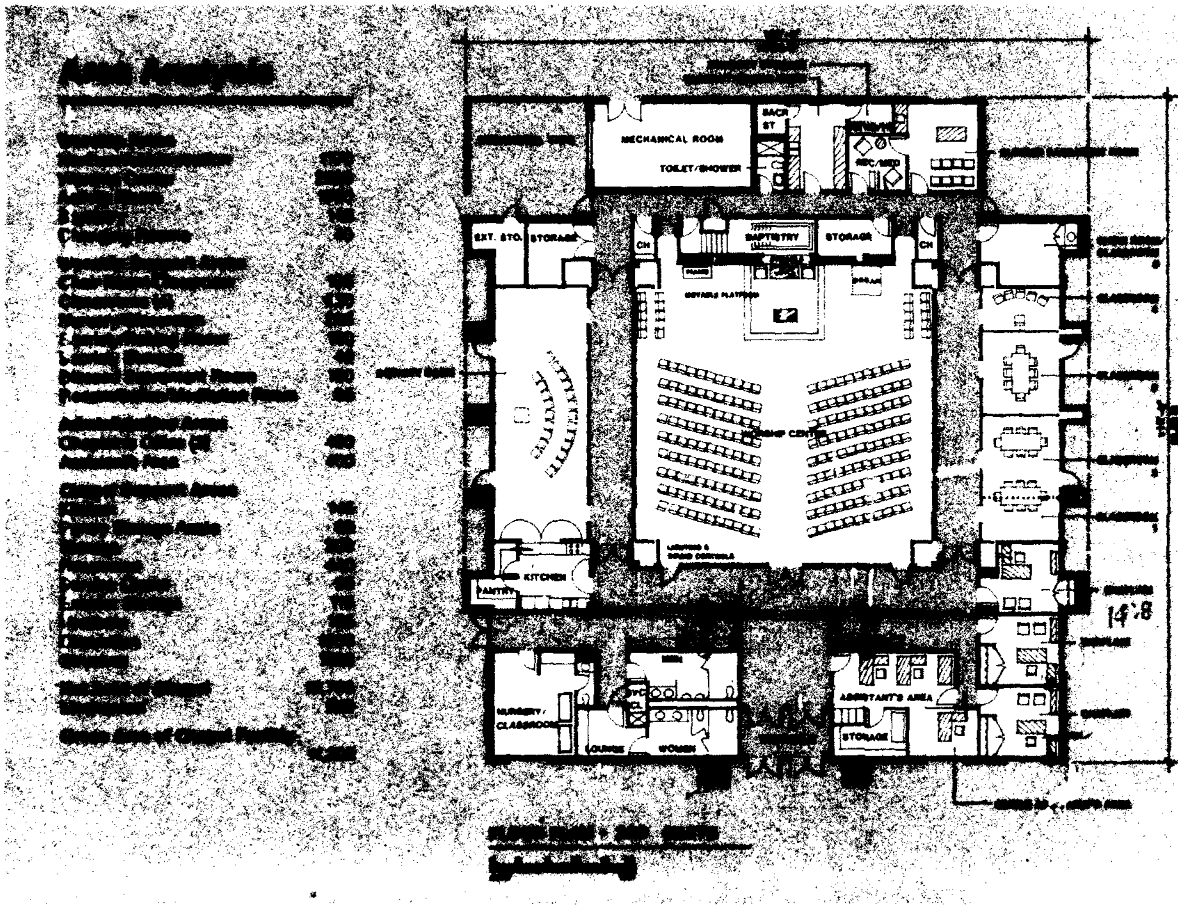
Introduction

The Facilities Standardization Program Army Unit Chapel provided the foundation from which the current design of the Fort Campbell Unit Chapel developed. The definitive design provided in the standardization program gives floor plans, functional layouts, construction material options, and other general design criteria. The user and designers decide where to take the definitive design in developing a unique, attractive facility. The fundamental concepts behind the definitive design include 1) the provision of worship seating with a variable capacity from two hundred to four hundred individuals, 2) flexible seating arrangements and variable room configurations using movable partitions, 3) the provision of flexible classrooms, meditation rooms, and gathering spaces also using movable partitions, 4) the servicing of all faiths without favor to a distinctive group. Figures 1 and 2 show the architectural rendering and the floor plan conceived for the Army Unit Chapel definitive design.

Criteria established by the Office, Chief of Chaplains, HQDA (DACH-AM), Design Guide, Chapels and Religious Education Facilities (June 1979 edition) was used in the definitive development. Although the size and general space arrangement of rooms relative to each other must remain constant during further development of the definitive, the simple geometric lines and the ability to vary construction materials, mechanical systems, and structural design make the facility readily adaptable to locations anywhere in the Continental United States.

[1] Structural Design Engineer, Engineering Division, US Army Engineer District, Louisville; Louisville, KY.

Figure 1. The architectural rendering of the Unit Chapel, Fort Campbell, Kentucky



and light control, an infinite number of arrangements are possible.

The organ console, while movable, is best located as shown here regardless of the seating pattern. Organ speakers will be concealed above the baptistry/storage area with their sound reflecting off of the ceiling plane. Choir seating may be added or deleted depending on the service. Regardless of the size of the assembly, a large number of variations are possible within this flexible building plan.

Ambient lighting will be accomplished with concealed cove fluorescent light strips on dimmers using deluxe warm-white lamps, semi-specular reflectors and remote ballasts. Accent and spot lighting will be handled from the header adjacent to the cove in a continuous bar-mounted theatrical lighting system. Theatrical lighting may also be installed at the perimeter of the central skylite, but must be designed to lower to floor level for maintenance and adjustments.

A high quality, low-level distributed loudspeaker system is most appropriate in this design to facilitate a relatively uniform distribution of sound

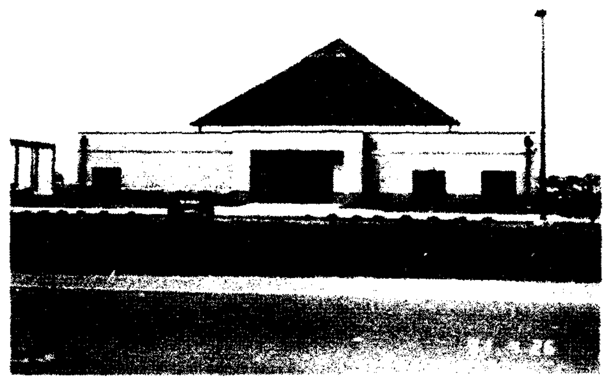
Figure 2. Floor plan, of the Unit Chapel, Fort Campbell, Kentucky

The Unit Chapel built at Fort Campbell, the first actually designed and constructed, provides an example to use in analyzing the effectiveness of a standard definitive design approach. Although provisions in the definitive alleviated architectural efforts in layout and schematics, specific interaction of structural elements had to be worked out in detail while maintaining the desired finished look.

Design Overview

The structural design of the Unit Chapel includes various elements and types of construction interfacing at key locations. The main structure is the central worship area framing. For Fort Campbell, the choice was finally made to go with a contractor-designed structural glued-laminated wood frame. A steel option was considered, but the user and designers preferred the look of the exposed glued-laminated wood. Steel-braced frames are used on the interior of the building as support for some of the movable partitions. Lateral loads are carried by tongue-and-groove wood roof decking in inclined diaphragm action and by bracing members. Timber purlins carry the wood decking, and a standing seam metal roof covers the wood deck on the exterior. The roof is capped with a steel-framed skylight at the apex of the wood frames. The surrounding low-rise facility support areas are of masonry load bearing shear walls with open web steel joists and metal decking for the roof support system. Non-structural partitions are metal studs with gypsum wallboard facing, and the exterior side walls adjacent to the activity room and classrooms do not carry axial loads so that future expansion remains possible and uncomplicated. The roof loads in the activity and classroom areas are carried by steel joists to other transverse masonry walls. Masonry parapets are used around the exterior to give the roof a flat look.

The foundation consists of various types of elements. Continuous wall footings designed for 3500-psf net bearing support the masonry load bearing walls. Isolated spread footings designed for 4000-psf net bearing support the tube steel columns of the interior steel braced frames, and belled piers designed for 6000-psf net bearing support the glued-laminated timber columns of the main frames. In addition, the presence of a shallow 72-inch diameter storm sewer running under the corner of the building required careful layout considerations and a reduction of the allowable net bearing pressure in a 20-foot vicinity around the storm sewer location to 2000 psf. A floating slab on grade reinforced with welded wire mesh is utilized for the floor system. Figure 3 shows front and rear elevations of the Fort Campbell Unit Chapel as constructed.



a. Front elevation

b. Rear elevation

Figure 3. Elevations of the Unit Chapel, Fort Campbell

Specific Design Challenges

The variation of structural elements and especially the presence of the glued-laminated wood framing created unique challenges in the development of the structural plans and specifications for this project. It was decided to develop the specifications to cover the glued-laminated wood framing as a contractor-designed item. With this approach, and because different features like the "clerestory" window and the movable partitions were performance sensitive, coordination during design and construction was very important. An expanded CEGS 6100 for rough carpentry was utilized in creating the specification for the glued-laminated wood framing. In addition to listing required structural properties for the wood, the main emphasis of the specification enhancement was the submittal of a detailed design analysis of the framing and its deflections.

Located in seismic zone 2, the Unit Chapel is exposed to earthquake motions, and a separation between the main structural system—the glued-laminated wood frames—and the surrounding masonry and steel framed construction had to be maintained due to potential differential movement. Cantilevered glued-laminated members and steel stud framing supported by the wood was kept independent of the functional area structure at the eaves with a 2-inch (horizontal and vertical) seismic joint. Also stated in the general notes of the contract drawings for the glued-laminated wood framing is a limiting lateral deflection of 1.2 inches for a basic wind speed of 70 mph.

To avoid operational problems with the movable partitions used in reconfiguring the worship center size, a limit on vertical deflection of 1/2 inch was required in the perimeter glued-laminated wood roof support beams spanning between the wood frames. These members support the movable partition track, and excessive deflection could cause the partition to bind. This problem was alleviated when the glued-laminated wood designer incorporated camber into the longer spans that supported the movable partitions. For the longest span of 68 feet, a camber of 3-1/2 inches was calculated as that required to counteract dead load deflection. Live load deflection was within the tolerable limit of 1/2 inch. The other long spans of 52.5 feet and 53.5 feet were given a camber of 2-3/8 inches and 2-1/2 inches, respectively.

The "clerestory" windows also complicated matters. *Originally conceived as a single* framed unit with ten 4-foot panels, the design had to be altered to include vertical slip joints at every other panel in the window frame due to the glued-laminated beam deflection under their weight. It was determined that 3/4-inch deflection would be seen under the load of the windows, and the original window frame design was incapable of absorbing that much movement. This interaction had to be coordinated between the window frame fabricator, the wood frame designers and the contractor, and was achieved during the shop drawing review process. Figures 4, 5, and 6 show finished details of the skylight at the apex of the wood frames, the movable partitions, "clerestory" windows, and the exposed glued-laminated wood framing.

In developing a definitive design, some site-specific circumstances can never be foreseen, and such was the case at Fort Campbell. There is an existing 72-inch storm sewer running through the site and actually directly under one corner of the Unit Chapel. The approximate location was known, but the exact position with respect to the planned building

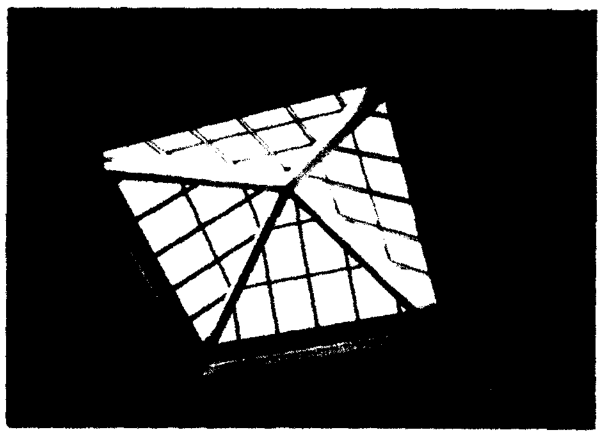


Figure 4. View from interior of skylight

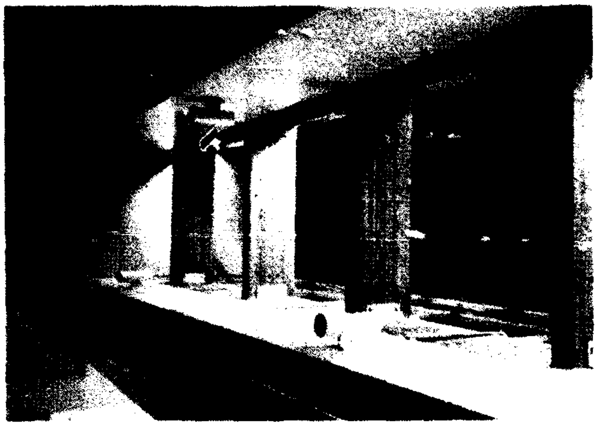


Figure 5. Exposed cantilevered glue-laminated wood framing

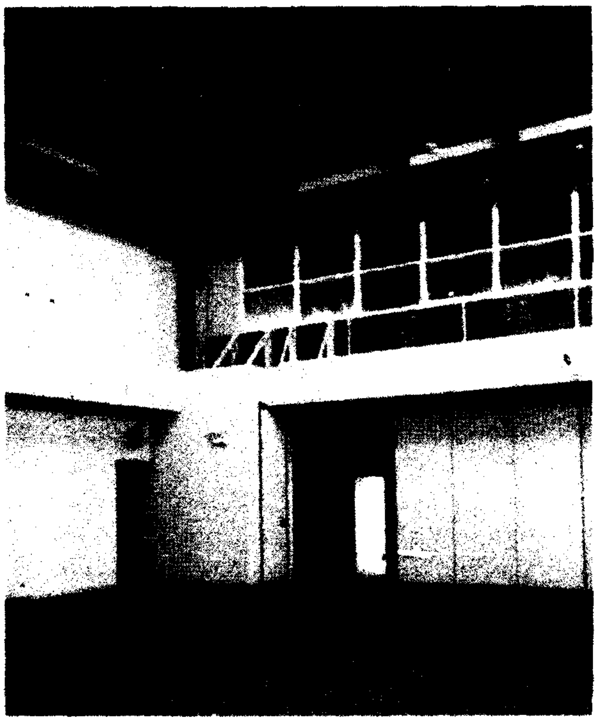


Figure 6. Moveable partitions, clerestory windows and exposed glue-laminated wood framing

layout was not. Originally, the sewer was to be relocated, but a cost savings of $100,000 was observed by leaving it in place and going to a pier foundation. To minimize the potential interference and surcharging of the sewer line, belled piers were used as foundation elements supporting the timber frames. This served to reduce the plan area of the foundation in comparison to isolated spread footings at the probable location of the sewer line, thus reducing risk of interference. Piers also carried the load deeper to a point below the sewer elevation which avoided additional load on it which would not have been accomplished with shallow, isolated spread footings. Strict attention and coordination had to be paid to this potential problem during the design and construction phases.

Conclusions

The use of a standard definitive design can be beneficial and cost effective if proper attention is given to critical details. Though conceived in a general context and developed to allow flexibility in the designers options, certain aspects and critical elements should be studied in detail in an attempt to identify potential problem areas. Any problem can be worked out, but too many problems to work out can be defeating of the purpose behind a definitive approach.

The most apparent deficiency in the definitive plans for the Unit Chapel is the lack of structural considerations in the conceptual plans. For example, the space allowed in the concept plans for structural members and mechanical equipment between the ceiling and roof system was inadequate for the required member depths and duct requirements at Fort Campbell. The roof lines were altered as required, as the definitive concept allows, but this action affected the architectural effect of the building's original geometry. Also, the considerations given to deflections and structure interaction were not identified until the design process was well underway, and, though not insurmountable, it would be highly advantageous to know these required considerations at the start. Obviously, specific requirements would not be known due to the possible variation in structure types, site locations, and loadings, but some foresight could be used in identifying and flagging potential special considerations.

References

US Army Corps of Engineers, Omaha District. Facilities Standardization Program, Army Unit Chapel.

US Army Corps of Engineers. 1989 (Jul). Fort Campbell Memorial Chapel, Contract Documents - Plans, Specifications and Design Analysis.

Effects of Interface Friction on the Behavior of Shallow-Buried Arches

by
Frank D. Dallriva, PE[1]

Abstract

The determination of the loads on and behavior of shallow-buried arch structures is complicated by the geometry of the arch and by the effects of soil-structure interaction. A common approach to the dynamic analysis of buried arches is to idealize the structure as a lumped parameter single-degree-of-freedom (SDOF) system. Input into the SDOF model includes a loading function and a structural resistance-deflection relationship. The load on a buried arch due to overpressure at the ground surface includes a radial and a tangential component. The radial component can be measured experimentally; however, it is extremely difficult to measure interface friction reliably.

Two reinforced concrete arches were tested statically in a sand backfill. The arches were semicircular with an inside radius of 1 ft, 9 in., and a thickness of 2 in. One arch was covered with two layers of 1/32-in.-thick Teflon at the soil-structure interface to significantly reduce the friction loads. The loading and behavior of the two arches were compared. Based on the experimental data, it appears that interface friction on a shallow-buried arch has only a minimal effect on its ultimate capacity for the case of uniform static overpressure and sand backfill. However, there appeared to be a significant effect on the load-deflection relationship. The internal forces in the arch with lower friction tended more toward pure compression than those in the other arch.

Introduction

The objectives of this study were to evaluate the effects of soil-structure interaction, particularly interface skin friction, on the loading and behavior of a shallow-buried reinforced concrete arch structure in a sand backfill.

Two reinforced concrete arch structures were tested statically in a sand backfill at a depth-of-burial (DOB) of 7.5 in., which resulted in a depth-to-arch-diameter ratio of about 1:6. The arches were semicircular with an inside radius of 1 ft, 9 in., a thickness of 2 in. and were supported on continuous concrete footings. The arches were constr,ucted as nearly identical as possible except that one arch was covered with two sheets of 1/32-in.-thick Teflon at the soil-structure interface to significantly reduce the interface skin friction. Since the frictional force is difficult to measure reliably, the use of Teflon on one arch provided a means of evaluating the effects of interface skin friction by

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comparing the test results with the arch without Teflon. The first arch tested, which did not have Teflon at the soil-structure interface, was designated as arch S-1. The arch with two Teflon layers at the soil-structure interface was designated arch S-2.

Experimental Investigation

Arch construction details

Construction details and dimensions of the model arch structures are shown in Figure 1. The inside radius of the arches was 1 ft, 9 in., and the thickness of the arch rings was 2 in. Reinforcing steel in the radial direction consisted of D3 wire (area equals 0.0295 in.^{2}) spaced at 2-1/4 in. on center in each face, which resulted in a principal reinforcing steel ratio of approximately 0.008 in each face. Longitudinal reinforcing consisted of D3 wire spaced at 8.5 deg on center and was placed inside the radial steel. A concrete cover of 1/4 in. was maintained over the principal reinforcing steel. After concrete placement and removal of the forms, one of the arches received a 1/32-in. layer of Teflon. The Teflon was glued to the exterior surface and the edges of the arch ring.

Test configuration and procedure

Figure 2 shows the test configuration. The test device is capable of developing pressures up to 3,000 psi. Two layers of Teflon were placed on the inside face of the test chamber to reduce the amount of friction between the sand and the chamber. In each of the two tests, sand was placed to the proper height in the test facility in 6-in. lifts and compacted to provide a uniform support for the model structure. The precast concrete footings were set in place in the chamber, and a steel support for deflection gages was welded to embedded plates in the footings. The arch ring was then lowered into the chamber and placed in the proper position on the footings and grouted in. Transducers for measuring structure loading and behavior were then installed, and sand backfill was placed around and over the arch to a height of 7.5 in. above the arch crown. Steel endplates were used to close the ends of the arches. The ends of the arches and the steel plates were covered with Teflon to provide a Teflon-Teflon interface between the two, thereby reducing the effects of end support as the arches were loaded. After closing up the end of the arch, sand was placed in 6-in. lifts and compacted by making four passes with a 23-lb hand tamp having a 6-3/4- by 11-in. foot.

In each test the data type recorder was started immediately preceding the opening of the waterline valve used to fill the test device with water. The time required to fill the water chamber was approximately 20 minutes. A relief plug at the top of the water chamber indicated when the chamber was full at which time the waterline valve was closed to allow closing of the relief valve. The pump was then started and the pressure in the water chamber was increased very slowly to load the soil surface. As each test proceeded, a plot of water pressure versus arch-crown deflection was monitored to provide a means of determining when a terminate the test.

Instrumentation

Thirty channels of data were recorded on magnetic tape in each of the two tests on a 32-channel Sangamo Model III FM magnetic tape recorder. The data for each channel were later digitized, processed, and plotted. The instrumentation layout for both tests is shown in Figure 3.

Two water-pressure gages (Kulite Model HKM-375) were used to record the pressure applied to the soil surface over the arches. Two gages were used so that if one malfunctioned, data from the backup gage could be used. One of the water-pressure gages was used as a reference channel against which all other data were plotted.

Nine interface pressure (IP) gages (Micro-Gage Model P-302) were mounted around the arch ring at approximately every 22.5 deg to define the pressure distribution around the arch. The gages had a range of 1,000 psi.

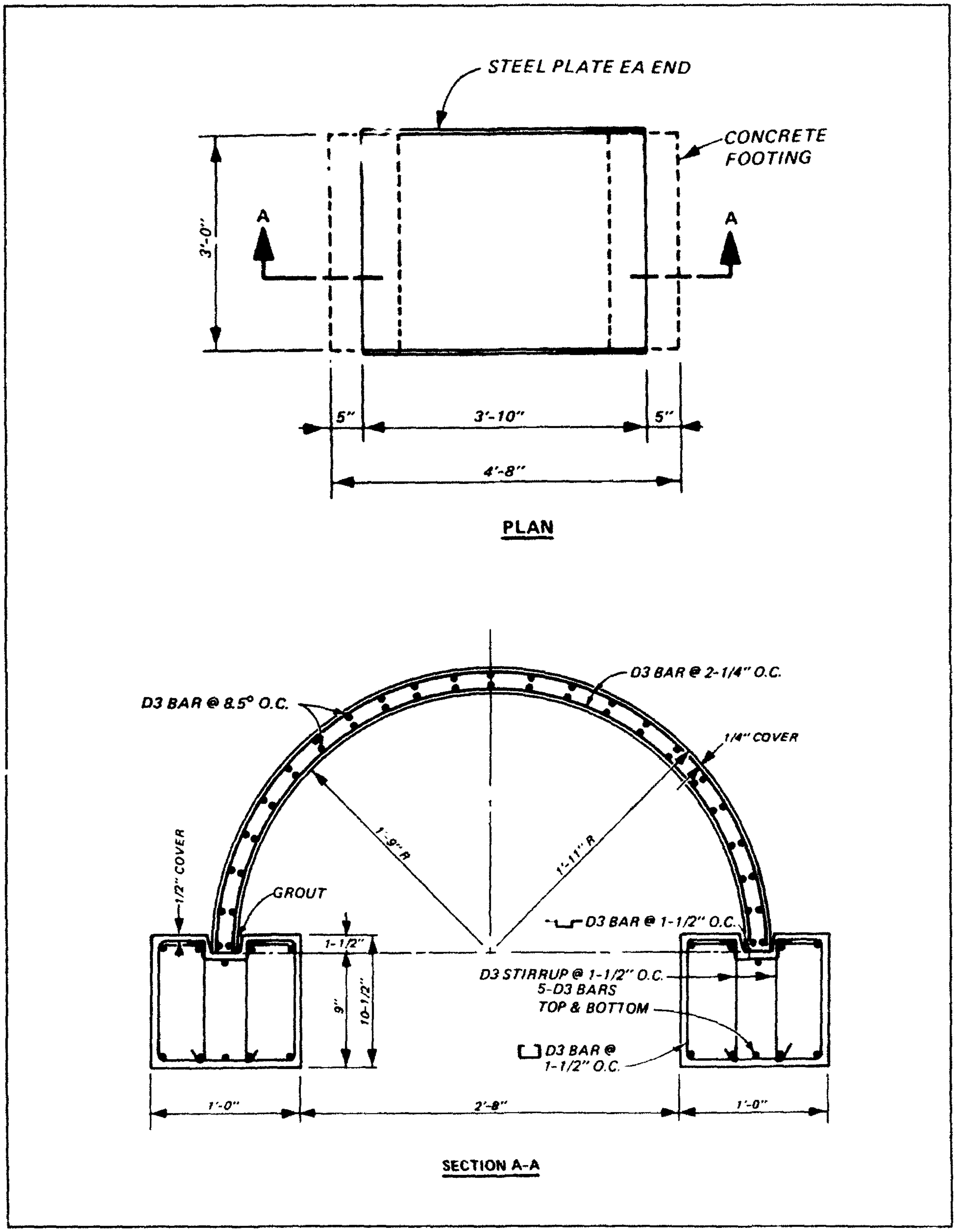

Figure 1. Construction details and dimensions

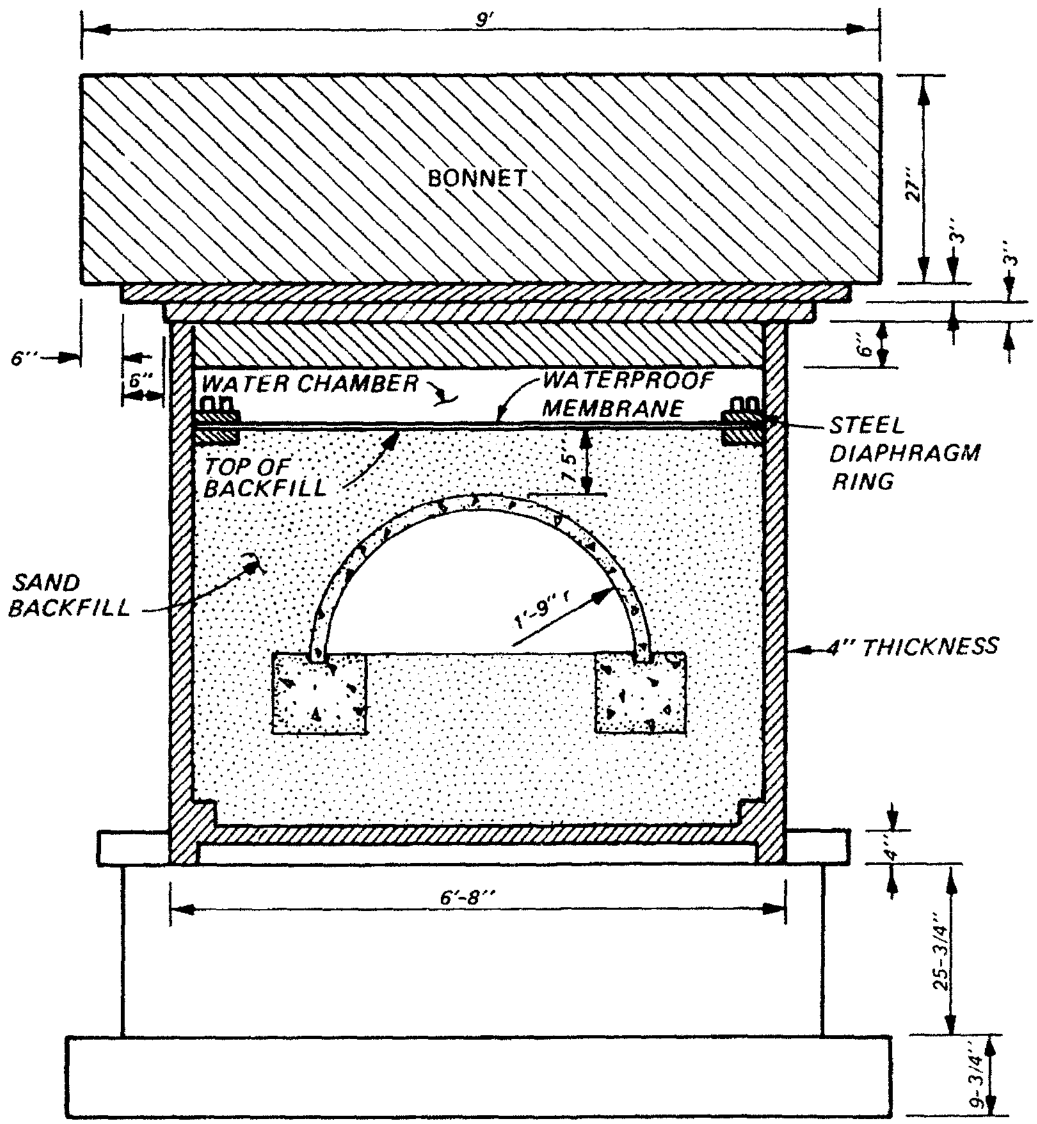

Figure 2. Test configuration

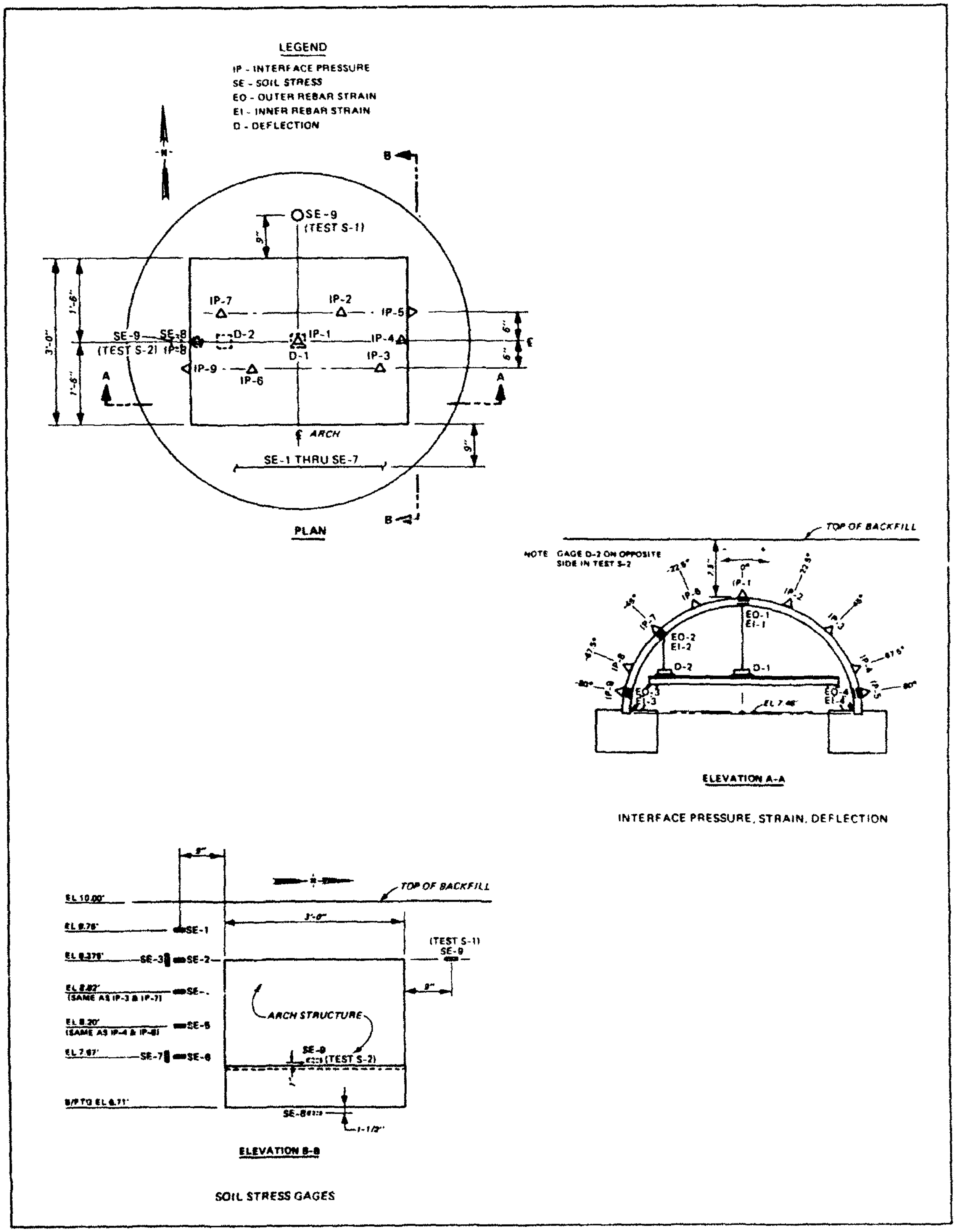

Figure 3. Instrumentation layout

The gages were installed in mounting hardware, which was fabricated at the US Army Engineer Waterways Experiment Station (WES), and installed in a sleeve that had been cast into the concrete arch ring. The gages were placed flush with the outside surface of the arch ring.

Eight single-axis, metal film strain gages were mounted to principal (radial) reinforcing bars in the arch ring. Four were on interior bars (EI) and four on exterior or outer bars (EO). These gages (Micro-Measurements Model EA-06-250BF-350-W) were 0.25-in., 350-ohm, temperature-compensated gages.

Two displacement (D) transducers (Celesco Model PT-101) with a range of 10 in. were used to record the vertical displacement of the arch ring at 0 and -45 deg. The body of the transducers was mounted to a steel support which was welded to embedded plates in the footings.

Nine free-field soil stress (SE) gages (Kulite Model LQV-080-8UH) were used in each test. Both vertical and horizontal stress measurements were made. In both tests, a gage was placed 1.5 in. below one of the footings so that bearing pressures could be measured. In test S-1 (no Teflon), two gages were used to measure vertical free-field stress at the arch crown elevation to provide a backup. In test S-2, instead of using a backup gage at the arch crown, one gage was placed 1 in. above the top of one of the footings to provide a measurement of the approximate interface pressure at the top of the footing.

Material properties

The concrete mix for these tests were designed to have a 28-day compressive strength of 4,000 psi. Cylinders were tested at 28 days and on the day of each arch test. The average 28-day concrete strength was about 4,400 psi.

One of the cylinders was instrumented with strain gages to allow the constitutive relationships of the concrete under uniaxial compressed to be evaluated. The computed modulus of elasticity was 4.35E6 and Poisson's ratio was determined to be 0.14.

All reinforcing steel used in the arch rings and footings was the American Society for Testing and Materials (ASTM) A496 D3 for deformed wire (ASTM 1969), which has a cross-sectional area of 0.0295 in. The wire was heat-treated in an oven at WES until a yield strength of approximately 60,000 psi was reached. The yield strength before heat treatment was approximately 90,000 psi. Random samples of the reinforcement were tested to rupture in a tensile-testing apparatus. The yield and ultimate strengths of the reinforcement were computed by dividing the applied tensile force by the original cross-sectional area of the bar. Results of the reinforcement tensile tests and the concrete compressive tests are presented in Table 1.

Table 1
Concrete Compressive Test and Steel Tensile Test Results

Reinforcing Steel			
Wire Size	Sample	Yield Strength, psi	Ultimate Strength, psi
D3	1	65,830	75,900
D3	2	65,400	75,000
D3	3	67,500	75,900
D3	4	57,030	65,770
D3	5	57,130	64,830
D3	6	57,630	65,570
D3	Average	61,770	70,550
Concrete			
Batch	28-Day Compressive Strength, psi	Compressive Strength on Test Day, psi	Element Age When Tested, days
1	4,460	4,420 (S-1)	35
1		4,560 (S-2)	45
2	4,320	4,390 (S-1)	35

The sand backfill was obtained locally from a commercial supplier. The sand was classified as poorly graded (SP) according to the Unified Soil Classification System (US Department of Defense 1968). Laboratory

tests were conducted on samples of the sand to determine its gradation, compaction characteristics, and angle of internal friction, which was 38.5 deg. As the backfill was being placed in 6-in. lifts, several water content and density readings were taken at each lift using a Troxler nuclear testing device. The water content measurements in test S-1 ranged from 4.9 to 12.4 percent and averaged 7.4 percent, while in test S-2 they ranged from 2.7 to 7.6 percent and averaged 5.9 percent. The dry density measurements in test S-1 ranged from 101.6 to 108.3 pcf with an average of 105.4 pcf. The dry density measurements in test S-2 ranged from 99.2 to 109.1 pcf with an average of 104.1 pcf.

Experimental Results

Observations

As the test of arch S-1 proceeded, water pressure versus crown deflection was closely monitored. When the water pressure reached about 550 psi, the crown deflection was only about 0.35 in. To continue loading the arch at this point would likely have resulted in clipping any additional data since the calibration steps on most of the gages were set at 500 psi. Therefore, the water pressure was lowered back to zero, and the arch was visually inspected through a borescope. The only indication of any damage was some very small hairline cracks at the arch crown. At this point, it was decided that the calibration steps would be reset to a higher value and the structure would be reloaded. The water pressure was again slowly applied until it reached about 840 psi. At this point, it was determined that the structure should be inspected for damage in case the deflection plot being viewed was in error. To view the arch through the borescope requires that the arch be unloaded first. If the arch were to fail catastrophically, water and sand under high pressures could be forced out through the opening used for the borescope, possibly resulting in serious injury to anyone looking through the borescope. The water pressure, therefore, was decreased to zero. The attempt to view the damage was not successful, because rigid body displacement and footing rotation had caused sand to block the end of the borescope. At this point, it was decided to reload the arch. The water pressure was again applied and reached approximately 700 psi, at which time a decrease in pressure and an increase in crown deflection indicated failure of the arch. The test was terminated at this point.

The knowledge gained in testing arch S-1 resulted in better gage predictions for arch S-2. In this test only one loading sequence was conducted. The water pressure was applied slowly until a large increase in crown deflection and a decrease in pressure occurred at about 820 psi, which indicated that the arch had failed. After the pressure had been lowered to zero, visual inspection of the inside of the arch was made using the borescope. This inspection revealed that severe damage had occurred. The test was terminated, and the structure was excavated.

Structural damage

Posttest photographs of arch S-1 are shown in Figures 4 and 5. The crown deflection relative to the footings was about 2-3/4 in. The average rigid body displacement, obtained by averaging the downward displacement of the footings at the four corners of the structure, was about 1/4 in. Each of the footings rotated inward about 1.5 deg. Small tension cracks occurred on the outside and crushing and spalling occurred on the inside of the arch ring at about +45 and -45 deg and extended the entire length of the arch, indicating flexural behavior at these locations. On the west side, a large crack ran the length of the arch. At the north end, the crack was located at about -30 deg and it ran downward in a zigzag pattern to the other end where it ended at about -80 deg. Some local buckling of the reinforcing bars was evident at this crack. At the north end of this crack the arch ring appeared to have sheared through and slipped to where the two sections were overlapping. Small tension cracks were visible on the interior of the arch at the crown, and a small amount of concrete crushing was visible on

Figure 4. Posttest view of north end of arch S-1

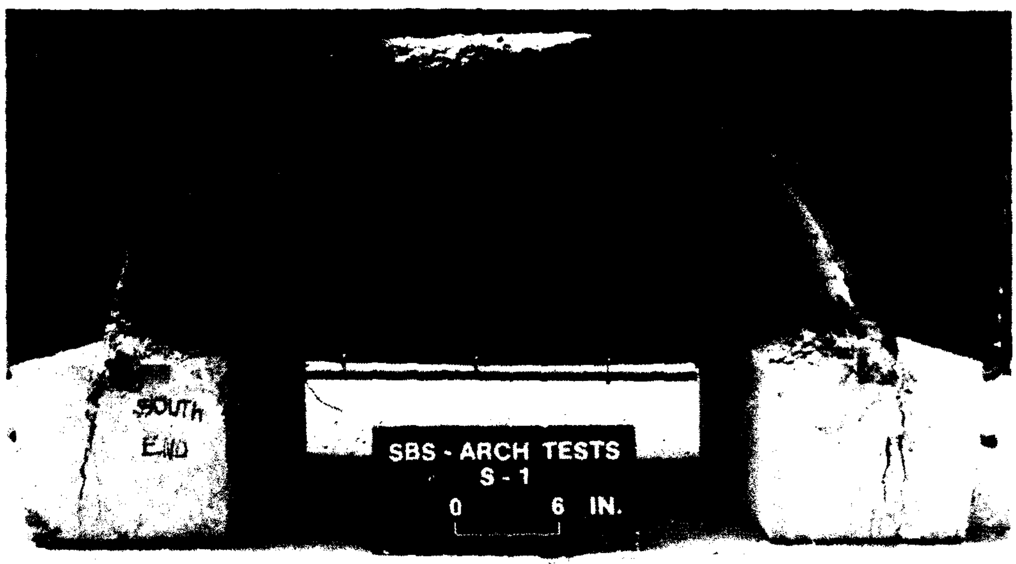

Figure 5. Posttest view of south end of arch S-1

the exterior, *indicating flexural displacement* at the crown. Near the arch-footing intersection, major concrete cracking and spalling occurred on both sides and the arch was cracked all the way through its thickness. Large cracks in the footing ran from directly below the arch ring to the bottom of the footing.

Posttest damage photographs of arch S-2 are shown in Figures 6 and 7. The crown deflection relative to the footings was 5-1/4 in. on the north end and 1 in. on the south end. The average rigid body displacement was 3/8 in. There was no measurable rotation of the footings. On the south end, the arch ring crushed and sheared all the way through its thickness for slightly more than one-half its length. A large radial crack formed and ran across the arch to about -20 deg where it turned and ran longitudinally to the south end of the arch. The arch segment, outlined by this continuous crack, deflected downward and was prevented from totally collapsing mainly by the reinforcing steel since most of the concrete was sheared through. At about +85 deg (east side) near the arch-footing intersection, concrete crushing and buckling of the reinforcing bars took place. Some crushing and spalling of the concrete also occurred on the west side near the arch-footing intersection; however, it was not as severe as on the east side.

Instrumented data

A comparison of the recorded arch-crown deflections is shown in Figure 8. Included in the comparison are the deflections corresponding to each load-unload sequence of arch S-1. When arch S-1 was unloaded, very little deflection was recovered. Upon reloading, the curve closely followed the unload curve up to about two-thirds the previous maximum load where it became less steep. In the third load sequence of test S-1 and in test S-2, the arch deflected suddenly, accompanied by a rapid pressure decrease due to the increase in volume in the test chamber. At this point the test was stopped. The curves show that the load-deflection relationship in test S-1 (first load sequence) and S-2 were very similar up to about 300 psi, at which point the slope of the curve for test S-1 became less steep, i.e., more deflection was occurring for a given increase in water pressure. The maximum pressure reached for both arches was slightly over 800 psi; however, the deflection of the arch S-1 was much greater at this pressure than was the deflection of arch S-2. It appears that this was not merely the result of unloading and reloading arch S-1, because the load-deflection curve for the initial loading shows arch S-1 to be less stiff than arch S-2. In both tests after the maximum load was reached, the slope of the curve is negative. This part of the load-deflection curve is not, however, an accurate measure of the static resistance. When the maximum load was reached in each test, the arch became unstable due to crushing of concrete, buckling of reinforcing bars, and shear. The response at this point became dynamic, in that the arch suddenly deflected a large amount. The change in volume in the test facility resulted in the pressure reduction. Due to the unstable nature of the behavior of the arch after the ultimate load was reached, it was impossible to record a true static load-deflection relationship. In reality, brittle or unstable behavior of this type would result in a sharp drop in resistance with very little additional deflection, perhaps followed by a less steep negative slope.

Analysis of Results

Experimental moments and thrusts in the arch rings were computed using strain data recorded from gages placed on the inner and outer reinforcing bars at 0, -45, and 80 deg from the crown. Comparisons made between the two arches included moment versus thrust, overpressure versus moment, and overpressure versus thrust.

A computer program was written to calculate the combined moments and thrusts in the arch ring using experimental strained data. The calculational procedure was based on the free-body diagram and the assumed linear strain distribution shown in Figure 9. The inner and outer experimental reinforcing bar strains were input, and a computed straight

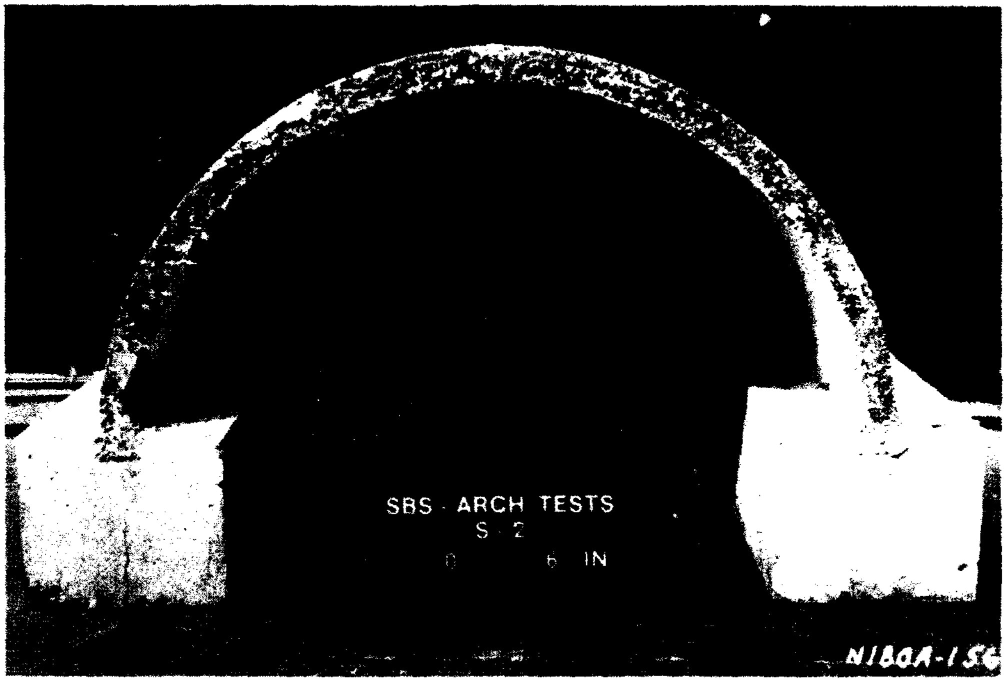

Figure 6. Posttest view of north end of arch S-2

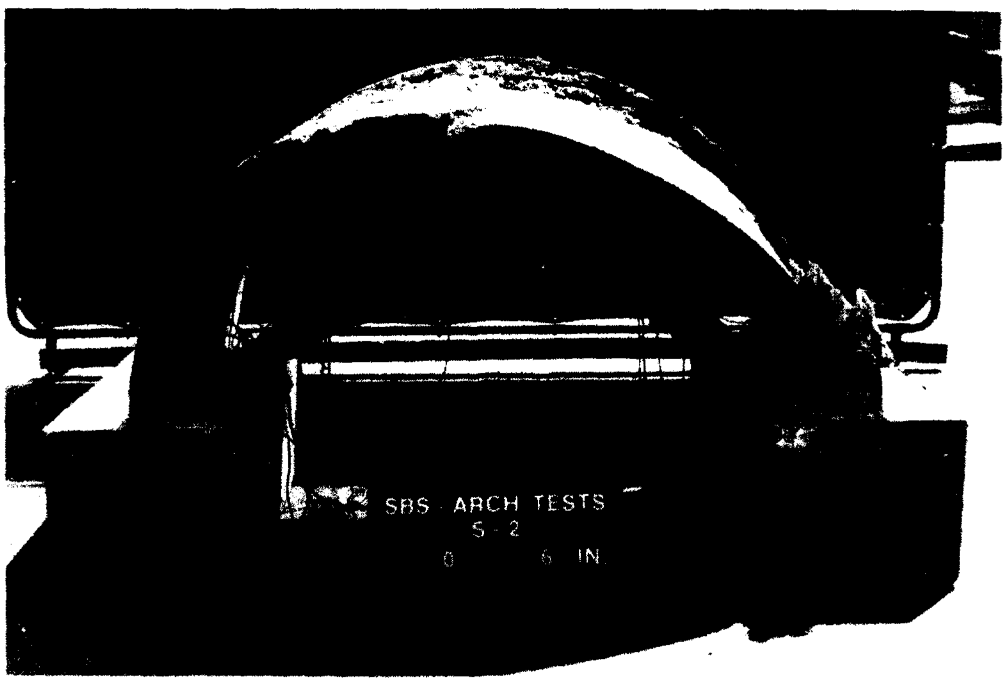

Figure 7. Posttest view of south end of arch S-2

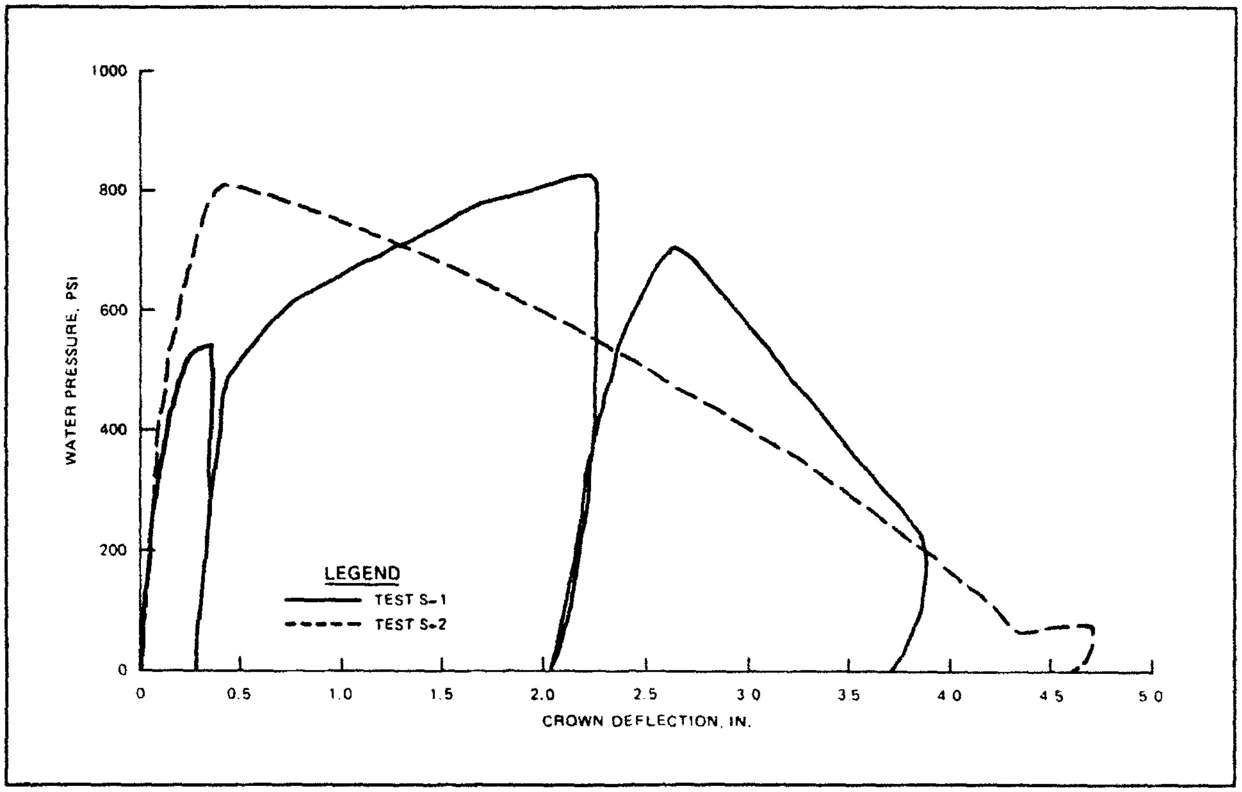

Figure 8. Measured arch crown deflection relative to footings

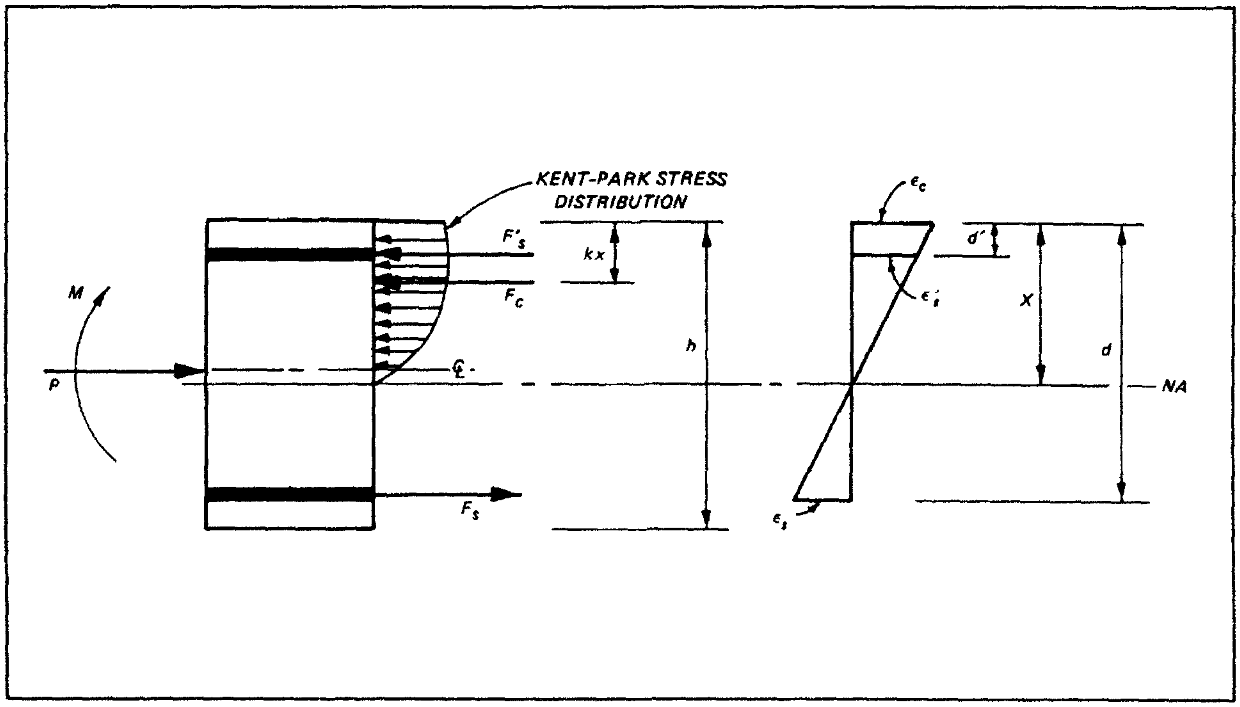

Figure 9. Concrete section free-body diagram and assumed strain distribution

line through the two points defined the strain distribution across the section. The concrete stress across the section was calculated based on the Kent-Park stress-strain curve shown in Figure 10 (Park and Paulay 1975).

The stress-strain curve used to compute the reinforcing steel stress is shown in Figure 11 and takes into account plastic strains. Once the stresses in the reinforcing steel and across the concrete section have been computed, the thrust and moment is calculated by

$$P = F_c + F_s' - F_s \qquad (1)$$

$$M = [(P)(h/2) + (Fu_{s})(d) - (F_{c})(kx) - (F_s')(d')] \qquad (2)$$

where the terms are graphically shown in Figure 9.

Figure 12 compares the load paths (moment-thrust interaction as the arches were being loaded) in both tests at each strain gage location along with the theoretical ultimate moment-thrust interaction diagram. The theoretical curve was computed assuming failure at a concrete compressive strain of 0.003. These plots show that at each of three locations in the arch ring, the ultimate strength of the section was reached in the compression regime of the moment-thrust diagram, i.e., above the point representing the condition of balance thrust. The location in both tests which had the highest ratio of thrust to moment throughout most of the loading was -45 deg with 0 deg having the lowest. Arch S-2 had higher thrust-to-moment ratios at all three locations than did arch S-1 throughout most of the loading, indicating that the behavior of the Teflon-covered arch was in more of a compression mode than the other arch.

Table 2 lists, for each of the three strain gage locations, the static overpressure when the theoretical ultimate capacity of the section was reached at that location. This was obtained by reading the thrust values from the load paths at the point where they intersected the ultimate moment-thrust curve and by finding the corresponding pressure on the thrust-pressure plots. In both test S-1 and test S-2, the theoretical ultimate capacity at the crown was reached first followed by the 45-deg location and then the 80-deg location. In test S-1 the overpressure, when the ultimate capacity at the crown was reached, was 390 psi, and in test S-2 it was 325 psi. In both tests, as the pressure increased after the ultimate capacity at the crown was reached, the moment started decreasing. In test S-2, the moment at the crown decreased to such an extent that the sign changed from positive to negative. This resulted in the moment and thrust at the crown decreasing below the ultimate capacity while the ultimate capacity was being reached at the other locations in the arch. Test S-1 appeared to behave similarly before the pressure was reduced to zero to recalibrate the gages. As the pressure increased after the ultimate capacity was reached at the strain gage locations, the moment at those locations tended to start reducing, resulting in the arch behaving more in compression. This had the effect of providing the arch with added load carrying capacity and was the result of the soil confining the arch ring.

Table 2
Water Pressure at Ultimate Strength

	Water Pressure, psi, at Indicated Locations in kArch Ring, deg		
Test	**0**	**-45**	**80**
S-1	390	410	420
S-2	325	530	580

Conclusions and Recommendations

Conclusions

Based on the thrust-moment plots computed from experimental strains and shown in Figure 11, the behavior of both arches was similar; however, the arch with the Teflon interface behaved slightly more in compression

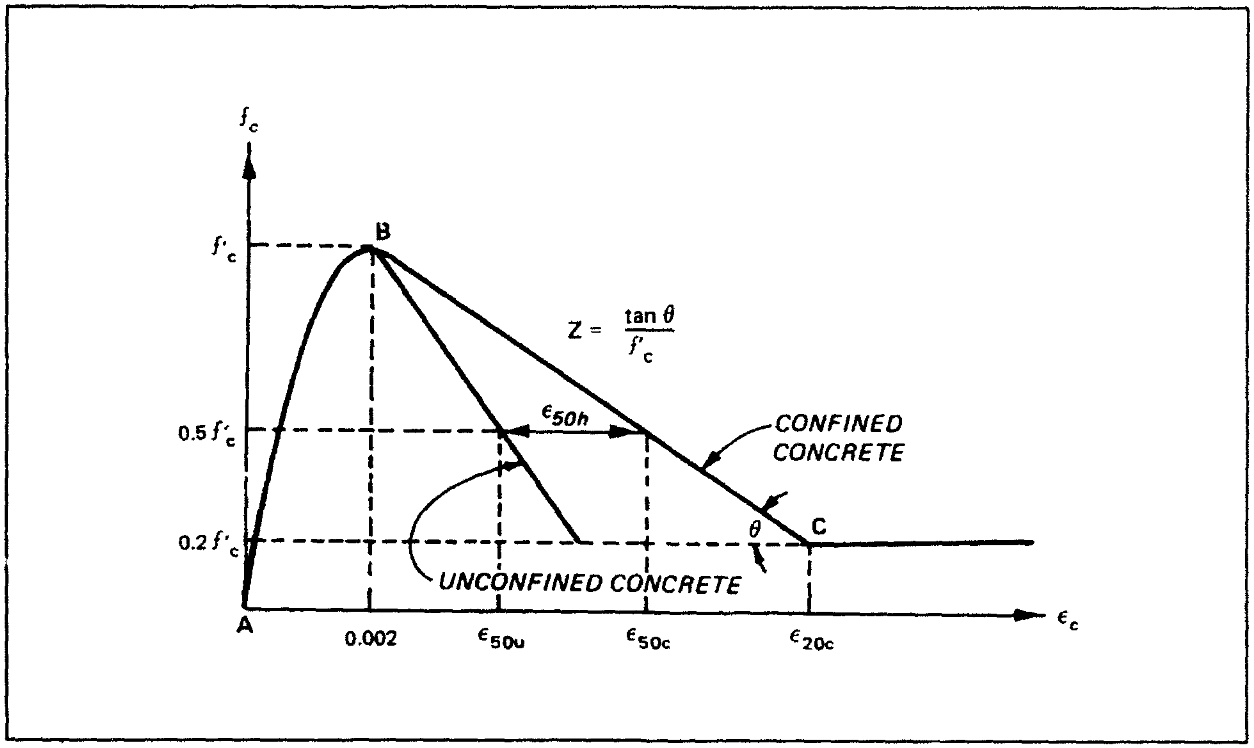

Figure 10. Kent-Park concrete model

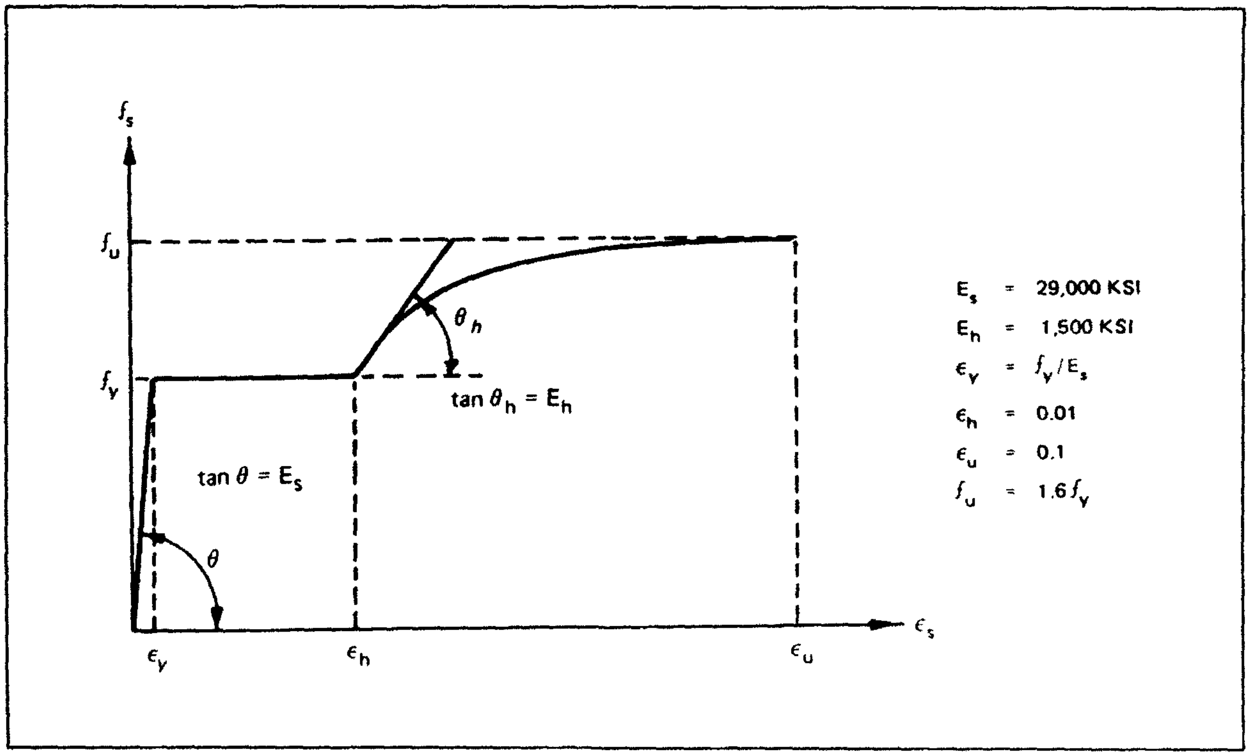

Figure 11. Reinforcing steel constitutive model

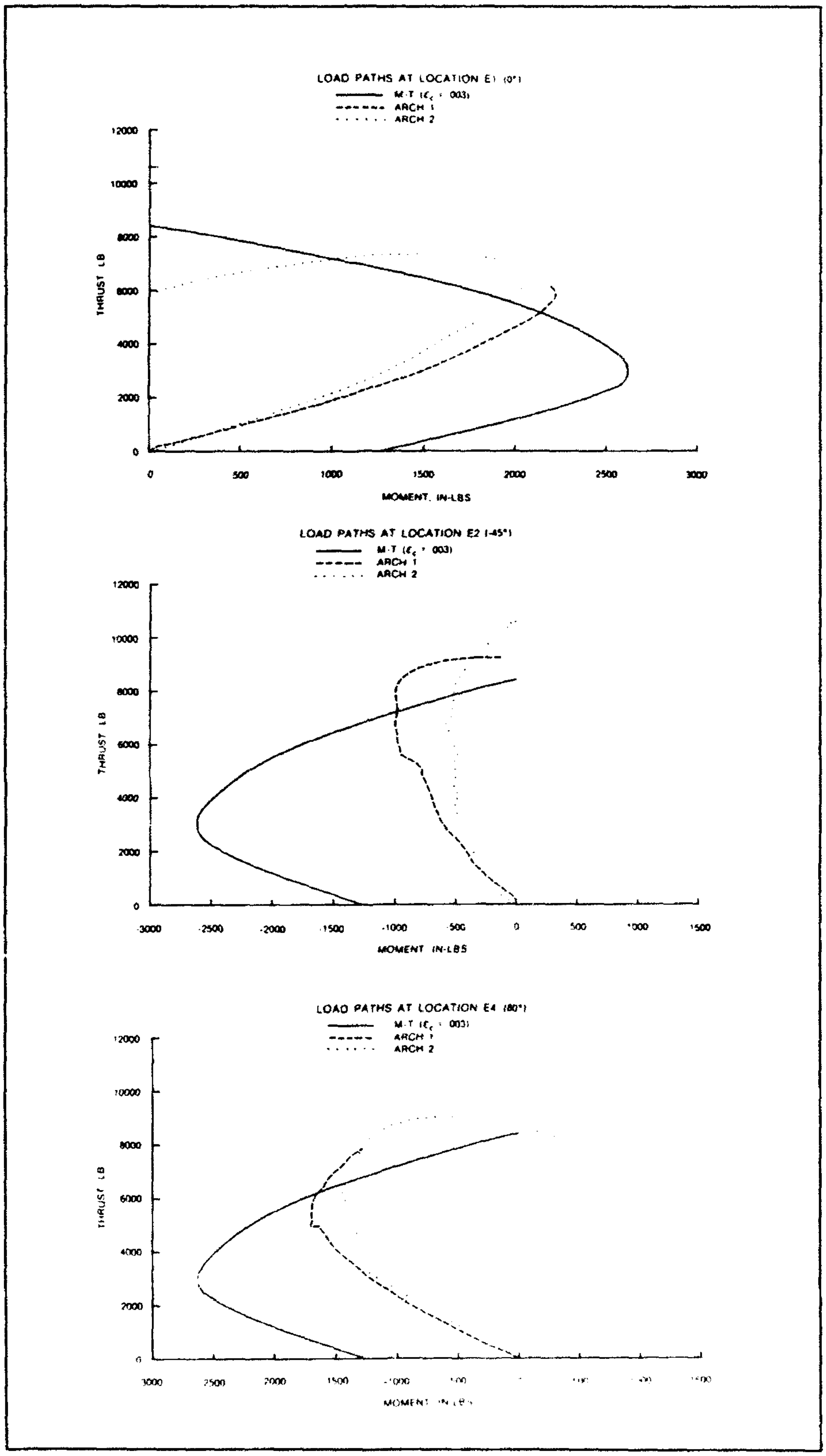

Figure 12. Load-path comparison between tests in each location

than did the other arch. Internal moments and thrusts were generally lower in the Teflon-covered arch. Although the maximum load in both tests was nearly equal, the higher moments in test S-1 resulted in a more gradual slope up to the maximum load in the load-deflection curve when compared to test S-2 (Figure 8). In test S-1 (no Teflon), the theoretical ultimate capacity of the concrete section at 0, -45, and 80 deg was reached at nearly the same overpressure, although the thrust-to-moment ratios were significantly different among the three locations. In test S-2, the ultimate capacity was reached first at the crown, followed by -45 and 80 deg, respectively, and the thrust-to-moment ratios among the three locations were signficantly different.

Rigid body displacement to the arches in the experiments could have contributed significantly to the soil-structure interaction effects, especially in test S-1, because the rigid body displacement accounted for a significant percentage of the total deflection at maximum overpressure. This indicates that the width of the footing could play an important role in determining the loading and resulting behavior of a buried concrete arch, especially in backfill material having high shear strength which allows more soil arching to occur.

Recommendations

An extension of the data base is necessary to support the findings of these two tests. Parameters which should be varied to investigate their effect on arch loading and behavior include soil backfill type, arch radius-to-thickness ratio, footing width, degree of fixity, reinforcement ratio, and the use of stirrups in the arch ring. Based upon the results presented herein, the following recommendations are made:

- Interface friction loading on shallow-buried arches should not be ignored since it appears to affect the load-deflection and the internal moment-thrust relationships.

- If the analysis procedure being used does not take into account the friction coefficient between the soil and the structure, assuming a fixed interface is probably better than assuming a frictionless interface.

- When using simplified analysis procedures to estimate arch behavior, soil arching should be taken into account using equations such as those by Flathau (ASCE 1984). The equation computes only the ability of the soil backfill to arch load from one point to another in response to a relative displacement between the two points. The analyst must determine whether or not soil arching will occur and, if so, whether it will be active or passive arching. Further experimental and analytical investigation should be conducted to determine the effects of pertinent parameters on soil arching as it relates to shallow-buried arches.

Acknowledgments

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References

American Society of Civil Engineers. 1984. "Design of Structures to Resist Nuclear Weapons Effects," Manual No. 42, Manual Subcommittee of the Committee on Structural Dynamics, Engineering Mechanics Division, American Society of Civil Engineers, Chapter 4, New York, NY.

American Society for Testing and Materials. 1969. "Standard Specifications for Deformed and Plain Billet Steel Bars for Concrete Reinforcement," *1969 Annual Book of ASTM Standards,* Part 4, Philadelphia, PA.

Park, R., and Paulay, T. 1975. *Reinforced Concrete Structure,* John Wiley and Sons, New York, NY.

US Department of Defense. 1968 (Jun). "Unified Soil Classification System for Roads, Airfields, Embankments, and Foundations," Military Standard MIL-STD-619B, Washington, DC.

Visual Display Papers

Replacement of Penstocks at Fort Peck Power Plant No. 1

by

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Abstract

Fort Peck Dam and Lake is the largest hydraulic fill dam in the world and the fifth largest man-made reservoir in the United States. Construction started in 1933 and was completed in 1940. Full hydropower was in operation by 1943. The Omaha District is tasked with the continued repair and maintenance to preserve such engineering achievements at Fort Peck Dam to assure continued operation. This paper centers on the evaluation of the existing riveted steel penstock system of Power Plant No. 1 and conclusions that led to a 19 million dollar penstock replacement project.

Introduction

Fort Peck Power Plant No. 1 is located on the Missouri River in northeast Montana approximately 20 miles southeast of Glasgow. The project consists of a hydraulic filled dam, two power plants, spillway, two flood tunnels and ancillary service areas, and buildings.

Power Plant No. 1 consists of a screened low-level intake supplying three Francis turbines via 5,400 ft of riveted steel pressure tunnel, trifurcation, and three penstocks (see Figure 1). The power tunnel is 24.67 ft in diameter. Two of the penstocks are 14 ft in diameter, and the third is 11 ft in diameter.

The present plant was constructed in stages, beginning with a single 35 MW unit in the early 1940's, then adding a trifurcation, surge tanks, a second 15 MW unit, and part of Penstock No. 3 in 1943. The third 35 MW unit was added in 1951. Each penstock has a butterfly valve positioned between the turbine and surge tank riser.

The design for the tunnel and surge tank was hydraulically model tested in 1940-41 for a maximum discharge of 9,500 cfs. This was to verify surge tank sizing and to determine waterhammer and hydraulic friction losses.

In 1952 testing was undertaken to determine the efficiency of Units 2 and 3. This testing indicated that the actual discharge of the units was significantly higher than the turbine model tests had predicted. Studies completed in 1975 indicated that the units should be uprated by rewinding the generator stators. This was subsequently done and the resulting generator ratings at full load were increased to 50 MW for Units 1 and 3, and 21 MW for Unit 2. This in effect further increased the discharge capacity of the turbines.

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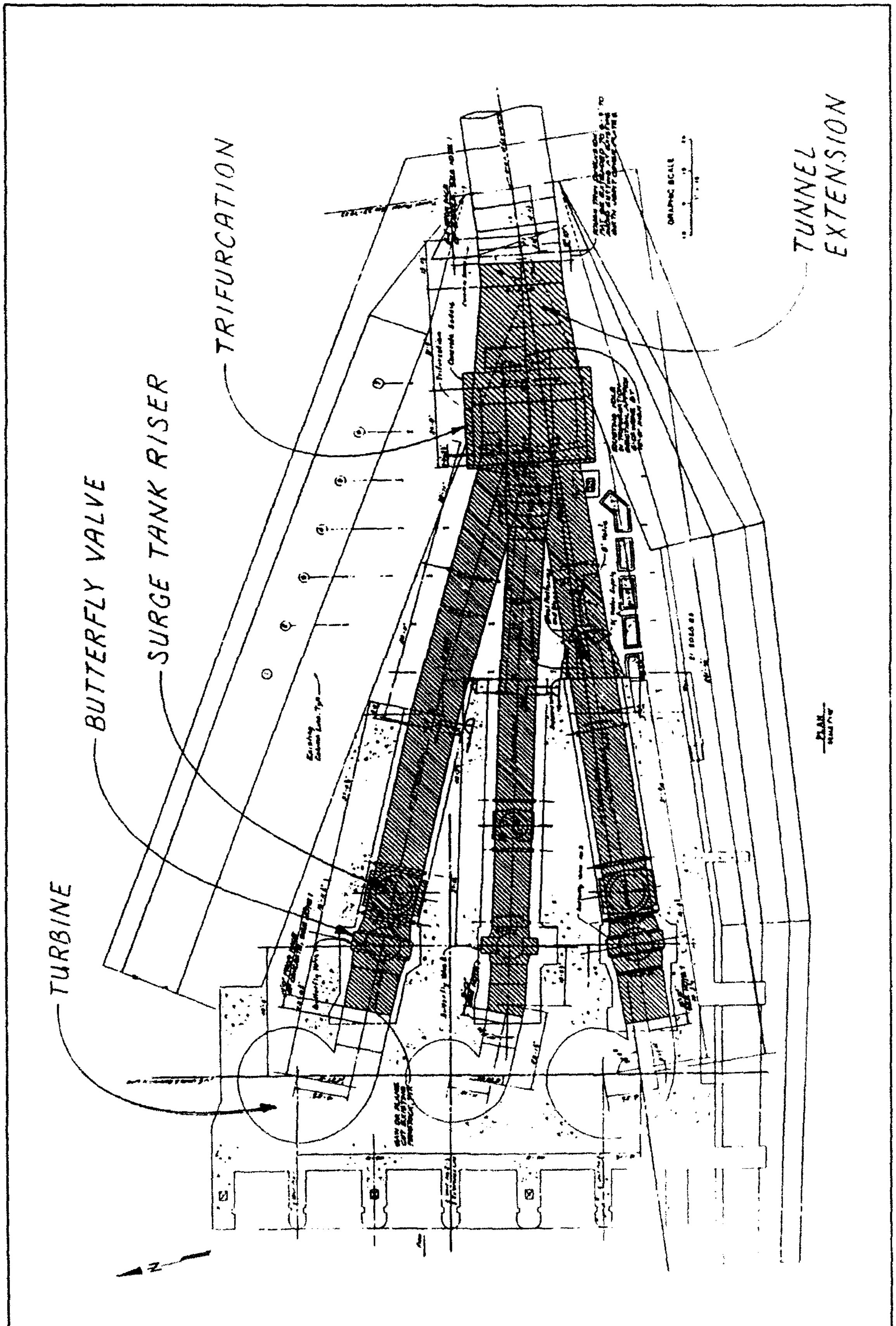

Figure 1. Existing penstock layout

Studies

Surge tank overtopping study

After the uprating, a question was raised concerning adequacy of the surge tanks with the increased discharge capacity of the plant. Computer modeling by the Omaha District indicated that the surge tanks could be overtopped. This study was followed with load rejection testing at the plant in October 1978. Extrapolation of the results of this testing confirmed that the surge tanks could indeed be overtopped. Discharge restrictions and loading rate limitations were imposed as a temporary solution. In order to regain full power benefits, the surge tank risers were computer modeled, and it was determined that further restricting the risers would solve the surge tank overtopping problem. This additional amount of restriction, however, would increase water hammer pressure in the penstocks.

A thorough inspection and preliminary analysis of the penstocks were conducted to determine the effect of increased pressure due to waterhammer. The results revealed that (1) extensive welding had been done on the penstock's riveted joints and rivet heads in order to stop leaks. (For the extent of the riveted steel construction, see Figure 2). No specifications or records could be found to document this welding; therefore, it was assumed to have been done under noncontrolled conditions, (2) the penstock system had no articulation joints to allow for differential settlement of the plant, thermal expansion/contraction, and contraction under pressure, (3) the penstocks were fabricated from a mixture of ASTM A-70, A-283, and A-285 steel under three separate contracts during the World War II era when there were extensive problems with steel allocations, and (4) the penstock system was currently operating with a factor of safety below 4 as required by EM 1110-1-2101 (Headquarters, Department of the Army 1963).

Shawinigan study

In July 1979, Shawinigan Engineering Corporation, San Francisco, CA, was contracted to evaluate the condition of the penstocks and recommend a permanent solution for the penstock and surge tank problems. In January 1981, the corporation submitted a final report. In the course of its study numerous steel plug samples and rivets were taken from the penstock system for chemical and physical analysis. A structural analysis using finite element analysis methods was conducted on the penstocks from the trifurcation to the turbines. The results of the testing and studies revealed low or questionable factors of safety for the penstocks based on (1) chemical and physical analysis of the rivets established that three and possibly four grades of rivets were used to construct the penstock system, and (2) chemical and physical analysis of the plate material indicated acceptable properties of the plates tested; however, a thorough testing program would be impractical since it would involve a great many samples due to the variety of plates used during construction, (3) structural analysis established that due to the lack of expansion/contraction joints in the penstock system, the rivets in the circumferential joints were underdesigned, for shear during normal loading, when a temperature variation of 35°F was considered. The lowest calculated factor of safety was 1.8, substantially below the required factor of safety of 4, based upon the ultimate tensile strength of the material, (4) due to the geometry and types of construction used for the trifurcation (see Figure 3), a rigorous analysis or testing would be required to determine an approximate factor of safety.

Shawinigan studied various alternatives for continued long term operation of the plant, such as increasing the height of the surge tanks to prevent overtopping. This, however, would not allow the discharge restrictions to be removed due to the newly discovered penstock problems. Three alternatives found that would allow the plant to be put back in full operations were (1) hoop-reinforcement on the existing penstocks, (2) pressure relief valve and bypass, and (3) replacement of the penstocks.

Conclusions were that hoop-reinforcement and pressure relief valves would relieve stresses and pressures; however, the integrity

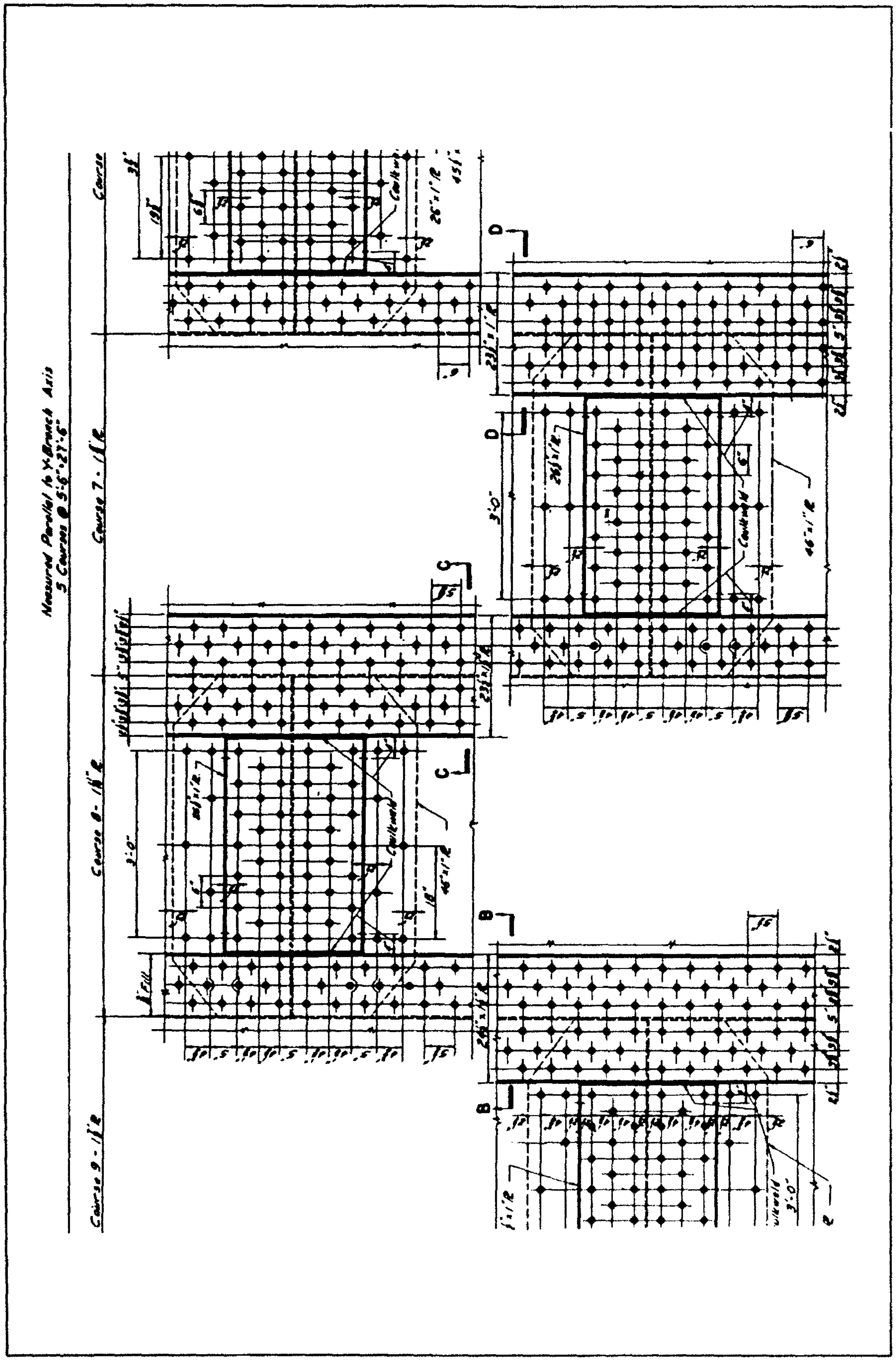

Figure 2. Existing penstock riveted joints

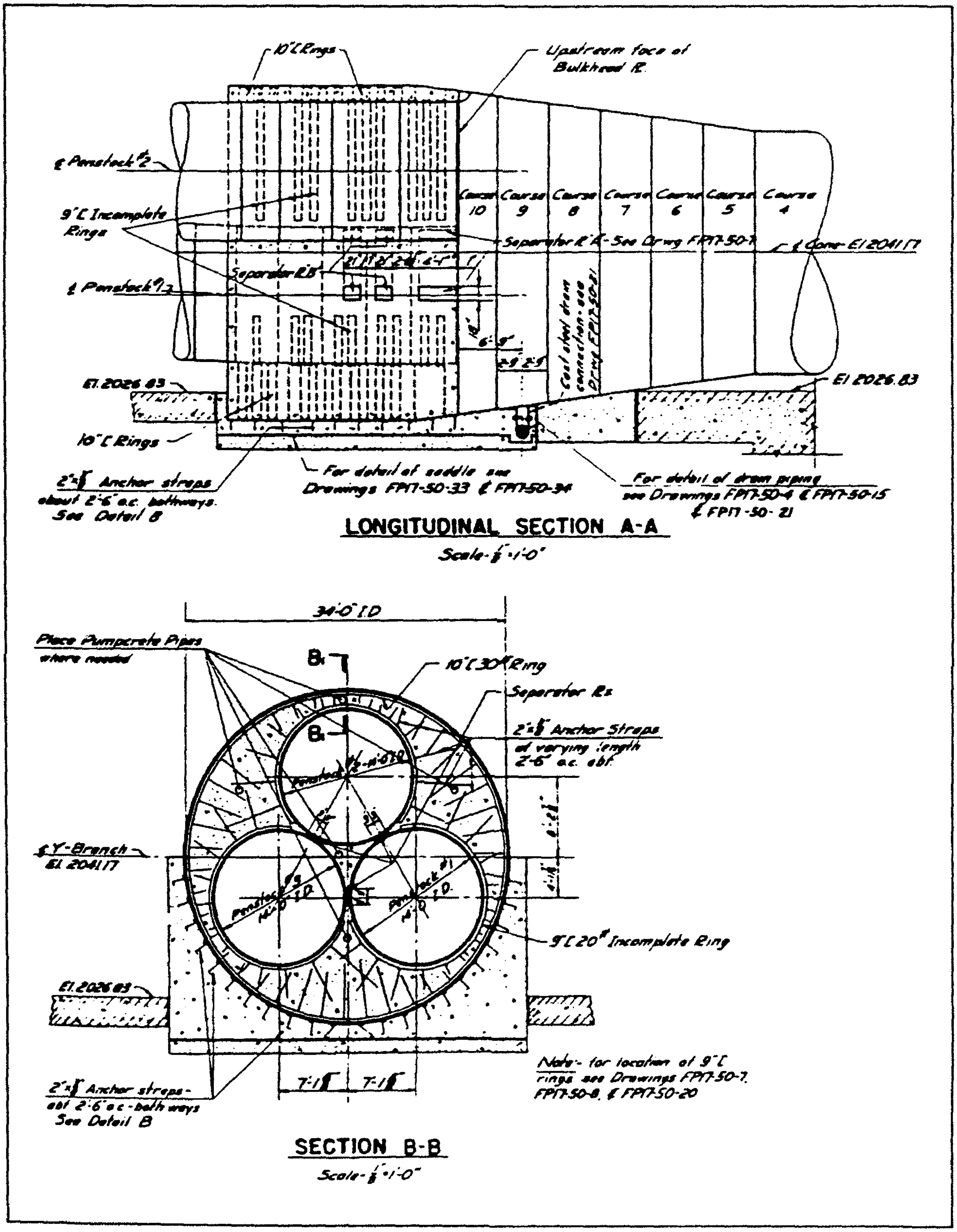

Figure 3. Existing trifurcation

of the plate material still was in question. It was determined that by necessity, replacement of the penstock system with new welded steel construction with expansion and contraction joints was required to allow the plant to be put back in full operation.

Reconnaissance report

In March of 1988, the Omaha District further analyzed the power tunnel from the intake structure to the tunnel portal at the power house and from the tunnel portal to the trifurcation.

Tunnel No. 1 was constructed in two consecutive stages. The first stage consisted of tunnel excavation, braced by ring beams, and subsequent placement of a 1.75-ft-thick concrete lining. The second stage consisted of placement of a 24.67-ft-diam, riveted steel liner with reinforced ring stiffeners and subsequent grouting of the annular void between the steel liner and previously cast concrete lining. Analysis was based in accordance with EM 1110-2-2901 (Headquarters, Department of the Army 1978). The steel liner and the concrete reinforcement were considered to act independently and then together to resist internal pressure. The lowest calculated factor of safety for the tunnel was determined to be 3.6. This was considered adequate as the rock mass surrounding the tunnel was assumed to not resist internal pressure.

The tunnel extension (see Figure 1) constructed of exposed riveted steel plate spanning between the tunnel portal and the trifurcation was also analyzed. It was found that rivet shear in the longitudinal joints controlled with the lowest factor of safety being 3.3. Replacement was recommended. Original specifications of the butterfly valves were examined and the factors of safety were found to be adequate.

In summary, the report recommended replacing the penstocks and tunnel extension. Additionally, it was recommended that trifurcation be replaced due to the structural integrity of connecting new welded construction to the riveted steel plate within the existing trifurcation. Also, the power tunnel had originally been coated with coal tar that was deteriorating, causing corrosion of the riveted steel liner. It was further recommended that during the outage the coal tar be removed and replaced with a vinyl paint system.

Plant Shutdown

In March of 1990, the Omaha District conducted an analysis of the effects of differential settlements on the existing penstock system. The powerhouse, trifurcation, and tunnel portal form distinct monoliths where differential settlements can occur. A geotechnical analysis predicted as much as 3/8-in. settlement between monoliths could have occurred over the 50-year life of the project.

The analysis was performed assuming 3/8 in.-settlement had occurred at an interior support of the most highly stressed penstock. It was found that this settlement could induce an additional bending stress of 15,370 psi on the most highly stressed joint. This bending stress and a corrosion allowance of 1/16 in. were added to Shawinigan's equation which considered normal penstock stress plus temperature stress. It was found that the factor of safety for the existing penstocks, considering differential settlement, can be as low as the following:

With current operation restrictions Internal pressure = 100 psi	F.S. = 1.2
With static head only Internal pressure = 84 psi	F.S. = 1.3
With emergency gate closed Internal pressure = 0	F.S. = 1.5

In addition, it was known that the powerhouse tended to tilt upstream during filling of the surge tanks, most likely causing further stress increases, and it was unknown how welding of the joints on the existing penstocks was influencing the factor of safety. It was determined that it was probable that the factor of safety was less than that shown above due to the indeterminate effects of powerhouse tilt and welding of the joints (see Figure 4).

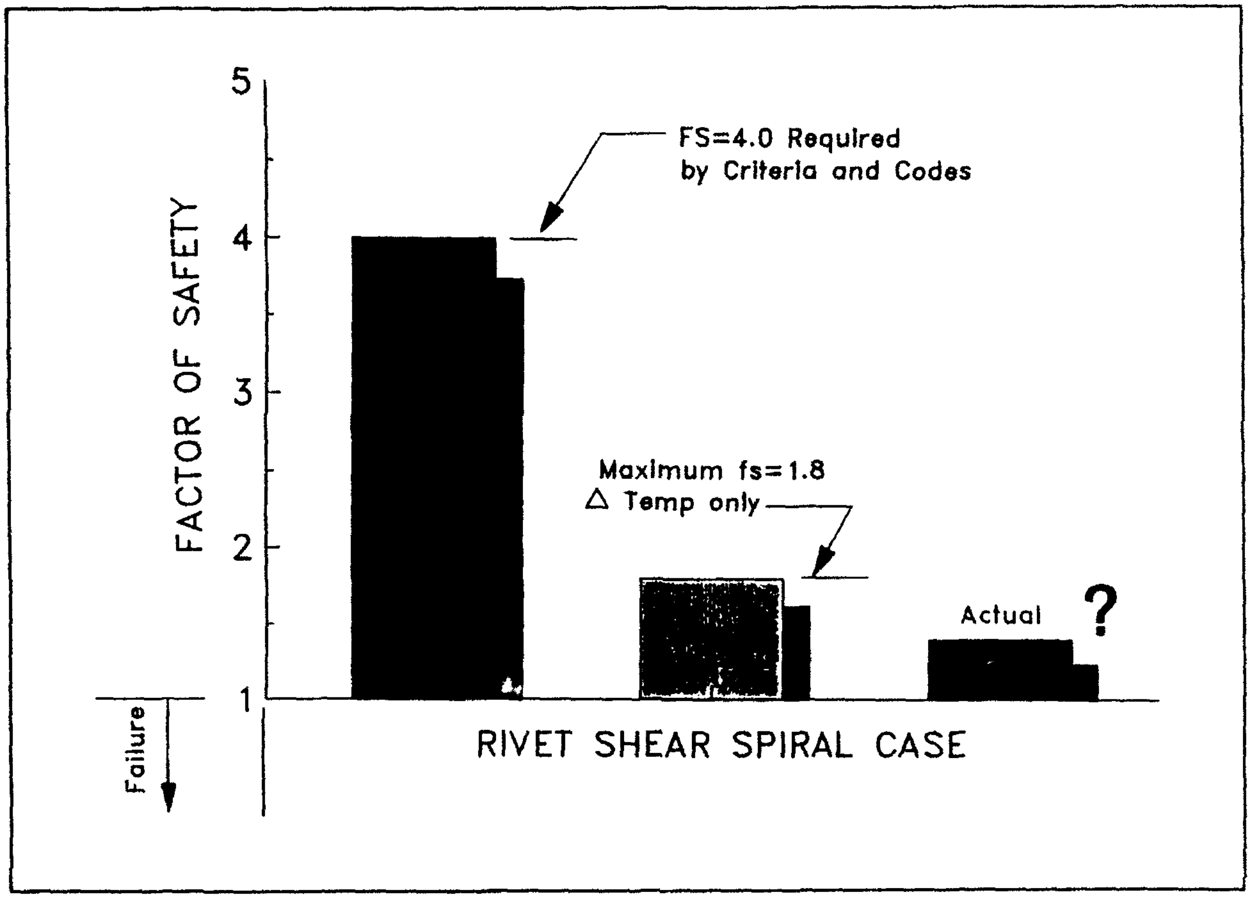

Figure 4. Existing penstock factor of safety

Based on the above safety issues, which indicated that a failure of the penstock system could be imminent, the plant was taken out of operation in April 1990.

Design and Construction Contracts

In April 1989, the A/E Firm of R. W. Beck and Associates, Seattle, WA, was awarded the contract to prepare plans and specifications for removal of the existing penstock and replacement with new welded steel construction (see Figure 5). They retained Sulzer-Escher Wyss, Zurich, Switzerland, as their subcontractor for the trifurcation design. All three penstocks were sized 14 ft in diameter, fitted with expansion/contraction joints, and supported on ring girders with rocker bearings to control any effects of differential settlement. Sulzer-Escher Wyss designed the trifurcation utilizing their patented design which is constructed with internal stiffener rings in the crotch, rather than the more conventional external stiffener rings (see Figure 6). Design stresses were determined by finite element analysis methods. The trifurcation was hydraulically model tested by the US Army Engineer Waterways Experiment Station (WES) to determine if the trifurcation was designed to the agreed upon hydraulic parameters. The testing verified the design met specifications.

During the design process, as funds became available, it was decided to replace the existing butterfly valves with new 14-ft-diam valves. This decision was based on the fact that after 50 years of service the valves most likely were reaching their design life. Replacement of the existing valves in later years, after the penstocks were installed, would require removal of portions of the new penstocks and trifurcation, and was not deemed economically feasible.

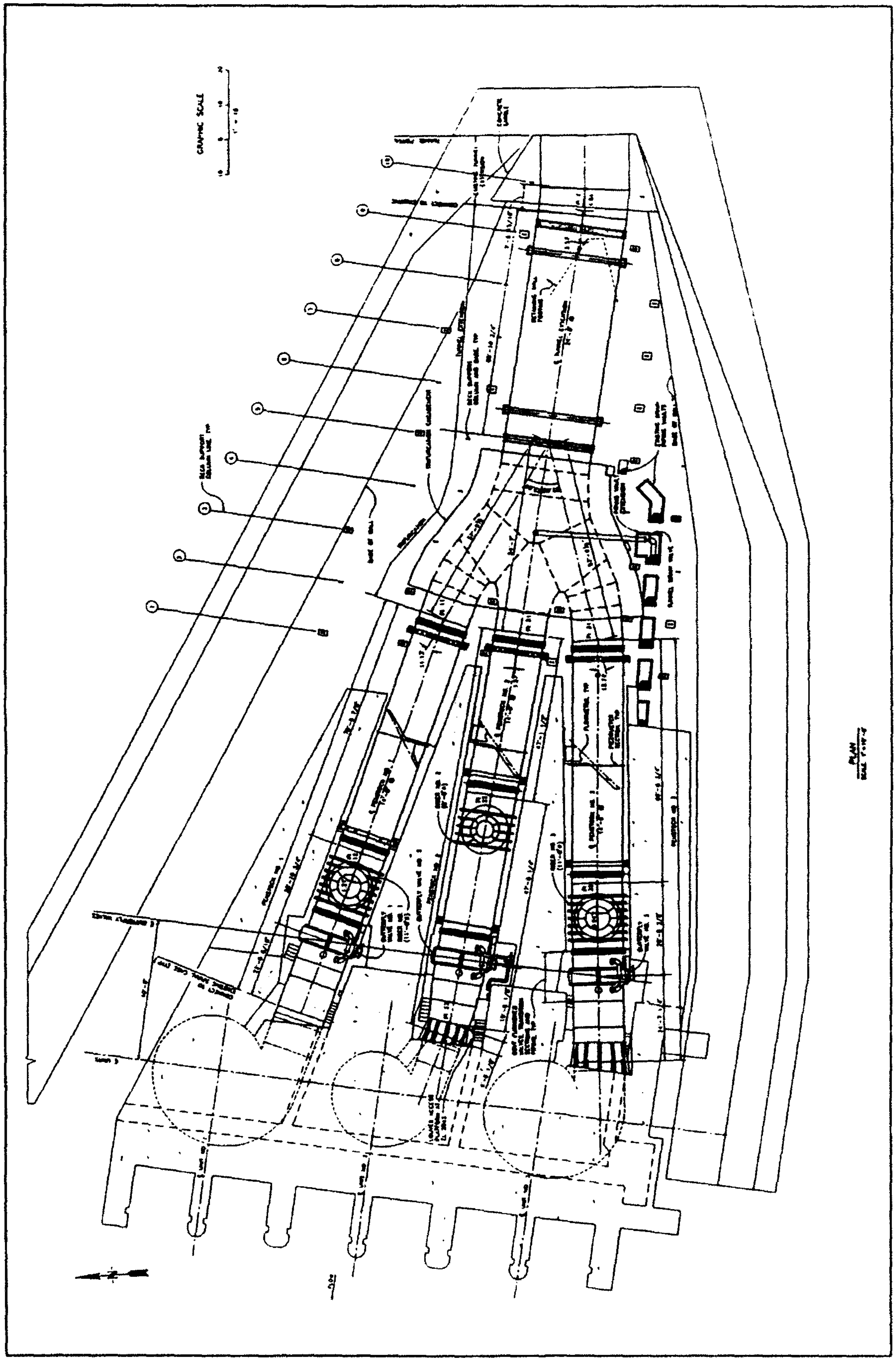

Figure 5. New penstock layout

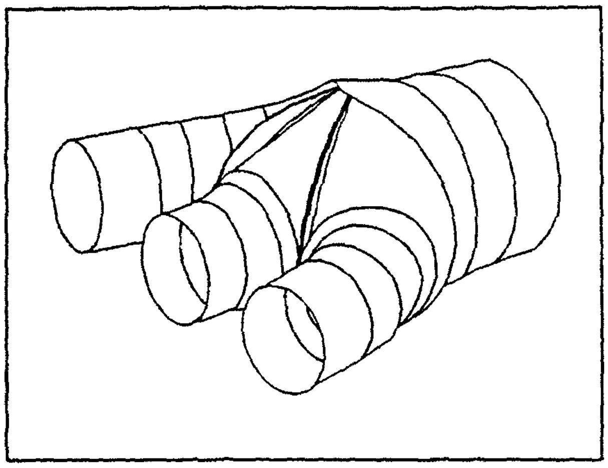

Figure 6. New trifurcation

R. W. Beck completed plans and specifications in August 1990. The following are the construction contracts awarded:

Butterfly valves	Kvaerner Hydro Power, Inc. San Francisco, California	$3,920,000
Tunnel Painting	Venture Construction, Inc. and Interstate Coatings Seattle, Washington	$2,702,000
Penstock replacement	Chicago Bridge and Iron Services, Inc. Fremont, California	$9,413,000

Design and construction, administrative, and contingencies put the total cost of the project at $19,047,000.

Currently, the project is on schedule for resuming complete operation of Power Plant No. 1 in November 1992.

References

Headquarters, Department of the Army. 1963 (Nov). *Working Stresses For Structural Design*, EM 1110-1-2101, Washington, DC.

Headquarters, Department of the Army. 1978 (Sep). *Tunnels and Shafts in Rock*, EM 1110-2-2901, Washington, DC.

Improved Strength Design of Reinforced Concrete Hydraulic Structures

by
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Abstract

Strength design (SD) criteria for reinforced concrete hydraulic structures (RCHS) have been based on a procedure that resulted in equivalence with the working stress design method given in Engineer Manual (EM) 1110-1-2101 (Headquarters, Department of the Army 1963). Recent studies have been directed toward reducing the conservatism associated with the working-stress-equivalent SD procedure, thereby improving the economy of RCHS.

Parametric studies indicate that an adjustment to the ACI 318-89 (ACI Committee 318, 1989) load factors results in close agreement with working stress designs for tension-control failures and for pure compression failures. These are the failure zones that are of primary interest for the design of RCHS. Also, serviceability is a primary concern for RCHS. Therefore, in addition to evaluating economics, the studies pertain to the effects of reinforcement grades and the effects of different steel ratios (as a percent of the balanced reinforcement ratio) on crack control and long-term behavior of RCHS.

Introduction

In general, existing reinforced concrete hydraulic structures (RCHS) designed by the Corps, using the working stress method of EM 1110-1-2101 have held up extremely well. The Corps began using strength design (SD) methods in 1981 to stay in step with industry, universities, and other engineering organizations. Engineer Technical Letter (ETL) 1110-2-265 (1981) was the first document providing guidance issued by the Corps concerning the use of strength design methods for hydraulic structures. The labor-intensive requirements of this ETL regarding the application of multiple load factors, as well as the fact that some load-factor combination conditions resulted in a less conservative design than if working stress methods were used, resulted in the development of ETL 1110-2-312 (Headquarters, Department of the Army 1988).

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The revised load factors in ETL 1110-2-312 were intended to assure that the resulting design was as conservative as if working stress methods were used. Also, the single load factor concept was introduced. The guidance in this ETL differs from ACI 318 primarily in the load factors, the concrete stress-strain relationship, and the yield strength of Grade 60 reinforcement. ETL 1110-2-312 guidance is intended to result in designs equivalent to those resulting when working stress methods are used.

The research discussed in this paper was performed in support of the development of a new Engineer Manual (Headquarters, Department of the Army 1990) for SD of RCHS. The Engineer Manual is currently in draft form and has been distributed to the Corps field offices for review. The new Engineer Manual modifies and expands the guidance in ETL 1110-2-312 with an approach similar to that of ACI 350R-89 (ACI Committee 350, 1990). The concrete stress-strain relationship and the yield strength of Grade 60 reinforcement given in ACI 318 are adopted. Also, the load factors bear a closer resemblance to ACI 318 and are modified by a "Hydraulic Structural Factor," H_f, to account for the serviceability needs of hydraulic structures.

Discussion

Reinforcing bars, availability

Although ETL 1110-2-312 specifically refers to the use of Grade 40 and Grade 60 reinforcing bars, it is not desirable to allow the use of a particular grade of reinforcement in the design phase if contractors are unwilling or not able to obtain that grade of steel during construction. Such a situation results in a short-notice redesign or postponement of the project.

The ACI Committee 439 (1989) gives a discussion of the physical properties and availability of steel reinforcement in the United States. This current assessment of availability points out that reinforcing bars rolled to the American Society for Testing and Materials (ASTM) A 615 (1987) specification for billet-steel are the most commonly specified and are available throughout the country. In general, the 60-ft length is the standard length available from most producers without special order. Rail-steel and axle-steel bars are not generally available in most areas of the country. The majority of construction uses billet-steel bars.

The ACI Committee 439 states that ASTM A 615 Grade 40 bars in sizes No. 3 through No. 6 and Grade 60 in sizes No. 3 through No. 11 are readily available in lengths up to 60 ft in all parts of the country. Bar sizes No. 7 through No. 11 in Grade 40 were deleted in the 1981 edition of ASTM A 615. Grade 60 bars in sizes No. 14 and No. 18 are generally available but are not usually kept in a fabricator's inventory.

Construction economy

An investigation was made to determine how sensitive construction cost would be to variation in the ratio of ρ_{max} to ρ_{bal} as the ratio varied from the ETL 1110-2-312 limit of 0.25 to the ACI 318 limit of 0.75. Beams varying in depth from 1 to 8 ft were designed for the maximum bending moment that could be sustained without needing compressive reinforcement. Two variations were obtained for each depth, one with reinforcement limited to one full layer and the other with as much positive reinforcement as would be needed to fully develop the strength without compressive reinforcement. In both cases, proper consideration was given to the constructibility requirements listed in EM 1110-2-2103 (Headquarters, Department of the Army, 1971). Two studies were made, one with $f_y = 48$ and one with $f_y = 60$. In every case, the results were normalized by considering the ratio of developed moment strength to estimated construction cost as the dependent variable and the ratio of ρ_{max} to ρ_{bal} as the independent variable:

$$\frac{\text{K–ft as a function of } \dfrac{\rho_{max}}{\rho_{bal}}}{\$\text{concrete} + \$\text{reinforcement}}$$

The design series with $f_y = 40$ is not reported in this paper because of the nonavailability of this steel grade; the series was run only for use with ETL 1110-2-312 and f_y comparisons and not for actual practical consideration.

Two major conclusions were reached: (1) Using more than one layer of reinforcement for a given required moment capacity is more costly than a beam of the same capacity with only one layer and should be used only where architectural considerations force the use of a more shallow beam regardless of the cost of that one beam, and (2) Moment strength per dollar increases as the ratio ρ_{max}/ρ_{bal} increases, up to a ρ ratio of 0.5, then stays constant as the ρ_{max} value increases up to the ACI 318 ductility limit of 0.75 ρ_{bal}. Thus there is no cost incentive to use a ρ ratio above 0.5, as well as the significant constructibility problems experienced as the ρ ratio increases above 0.5. The strength per dollar value can vary linearly as much as 45 percent as the ρ ratio varies from 0.25 to 0.5.

Crack control

Two major parameters that are limited by ETL 1110-2-312 for crack control are: (1) a maximum reinforcement spacing of 12 in. and (2) the use of a value of 48 ksi for reinforcement yield strength for Grade 60 reinforcement. ACI 350R-89 recommends the concept of sanitary durability coefficients for SD and states that the coefficients provide conservative service load stresses with Grade 60 steel. The coefficients were selected to provide crack control equivalent to that obtained with working stress design. Watertightness is assumed to be reasonably assured if (1) the concrete is well compacted, (2) crack width is minimized, (3) joints are properly designed and constructed, and (4) impervious linings are used where required.

ACI 350R-89 states that cracking can be held to a minimum by proper design, reinforcement distribution, and spacing of joints. It is preferable to use a larger number of small diameter bars for main reinforcement rather than an equal area of larger bars. A maximum bar spacing of 12 in. is recommended.

The load factors of ACI 318 are to be applied directly to sanitary structures and the total factored design load (U) increased by the sanitary durability coefficients. The load factors for both the lateral earth pressure and the lateral liquid pressure are taken as 1.7. The sanitary durability coefficients are (1) in calculations for reinforcement in flexure; the required strength should be 1.3 U , (2) in calculations for reinforcement in direct tension; the required strength should be 1.65 U , (3) in calculations for reinforcement in diagonal tension (shear); the required strength should be 1.3 times the excess of applied shear V_u less the shear carried by the concrete, ϕV_c. Thus, $1.3(V_u - \phi V_c) \leq \phi_\sigma V_s$, where $\phi_s V_s$ is the design capacity of the shear reinforcement, and (4) in calculations for the compressive region of flexure and compressive axial loads, and for all loads carried by concrete, the required strength should be 1.00 U .

Section strength

The major differences between the criteria given in ETL 1110-2-312 and that given in ACI 318 are presented in Table 1.

The parameters β , ε_{cmax} , and f_y are the primary parameters affecting the computed strength of a section, and ρ_{max} simply limits the amount of steel that the designer may use. Although the limit ρ_{max} is important, it does not directly affect the computation of section strength. The difference between the values of β in the ETL 1110-2-312 and ACI 318 criteria is actually a result of the ε_{cmax} values being different. Therefore, the most significant differences in the ETL 1110-2-312 and ACI 318 criteria for computing the strength of a section are the values of ε_{cmax} and f_y.

To aid in the parameter study, the strength of the section shown in Figure 1 (Liu 1981) is investigated. The computer program CSTR (Hamby and Price 1984) was used to aid in the development of the nominal strength design interaction diagrams presented in this discussion. The curves were digitized to allow

Table 1
Primary Parameters

Parameter	ETL	ACI
β	0.55 for $f_c' < 4.0$ ksi 0.55 for $f_c' > 4.0$ ksi	0.85 for $f_c' < 4.0$ ksi 0.85-0.5 (f_c' - 4.0) for $f_c' < 4.0$ ksi but not less than 0.65
ε_{cmax}	0.0015	0.003
P_{maxF}	0.7	0.8
ρ_{max}	0.25 ρ_b	0.75 ρ_b
f_y	40 ksi for Grade 40 48 ksi for Grade 60	40 ksi for Grade 40 60 ksi for Grade 60

Notes:
β = depth of stress block as a fraction of the depth of compressive face.
ε_{cmax} = maximum allowable concrete strain.
P_{maxF} = factor limiting strength due to axial load
P_{max} = maximum allowable reinforcement ratio given as fraction of balance ratio, ρ_b .
f_y = yield strength of reinforcement.

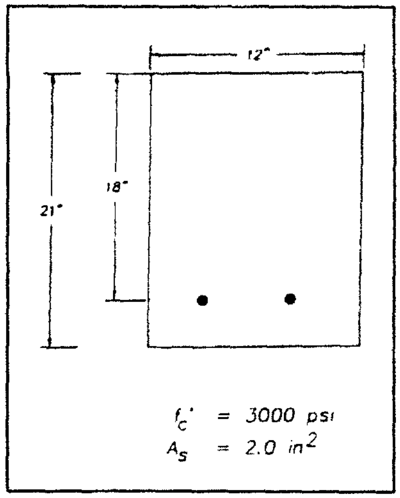

Figure 1. Singly reinforced beam

file manipulation for comparative plots of interaction diagrams. Also, some plots are the results of applying particular factors to the basic diagrams. Twenty-five complete plots were developed, but space limitations prohibit the presentation of all the plots. Some primary plots are discussed herein. A future Waterways Experiment Station (WES) technical report will present the study in more detail.

Figure 2 compares the ETL 1110-2-312 (ETL 1988), the ACI 318 (ACI 1989) and the ACI 350R-89 (ACI 1990) criteria for Grade 60 reinforcement. As discussed in the "Crack

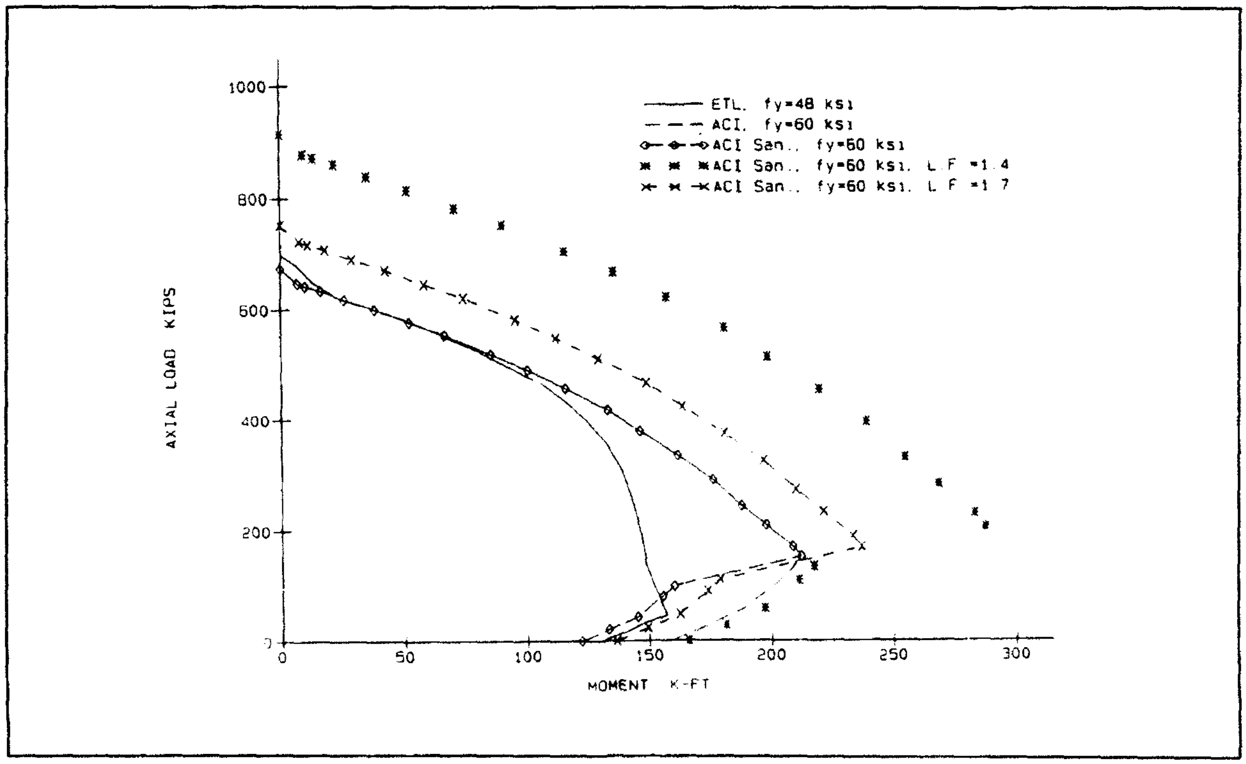

Figure 2. ETL, ACI, ACI Sanitary

Control" section of this report, the ACI 350R-89 durability coefficients require an increase in the strength required (1.3U) in the region below the balance point. Above the balance point, the required strength remains at 1.0 U. Since the axial-load/moment interaction diagrams may be thought of as failure curves, a division of the coordinates of points on the ACI 318 curves by 1.3 in the region below the balance point will reflect the effect of the sanitary durability coefficients. Figure 2 indicates that the ACI Sanitary criterion is more conservative than that of the ETL for values below the balance point and vice versa above the balance point when load factors are ignored. The ETL allows the use of a single-load factor for both dead and live loads. The ACI 318 and ACI Sanitary essentially require a factor of 1.4 for dead load and 1.7 for live load.

It is difficult for comparative purposes to determine exactly the value of an equivalent single-load factor for ACI 318 and ACI Sanitary. The lower the load factor, the less conservative the interaction curve will be. Figure 2 was developed with consideration of the load factor parameter when $f_y = 60$ ksi. Since 1.4 and 1.7 are the lower and upper bound load factors, respectively, they are reflected in Figure 2. A load factor of 1.9 is assumed for the ETL curve; therefore, the ACI Sanitary curve was multiplied by 1.9/1.4 and 1.9/1.7 for the load factors of 1.4 and 1.7, respectively. These curves are also shown in Figure 2.

ACI 350R-89 does not require a reduction in strength or an increase in load to minimize cracking in the region above the balance point. In contrast, the ETL criteria do not differentiate between the compression control and tension control regions for crack control and serviceability. The ACI Sanitary approach has some validity because tensile cracking is more likely to occur in the region below the balance point.

The consideration of load factors (1.4 and 1.7) in Figure 2 indicates that the ACI Sanitary criterion is considerably less conservative than that of the ETL for the region below the balance point, except for loading conditions where the live load is large compared to the dead load. Figure 2 indicates that the ACI Sanitary criterion with a load factor of 1.7 is similar to the ETL criterion below the balance point, but will improve the economy of structures in the region above the balance point until the region of large axial loads where the criteria give similar values. As previously discussed, the axial compressive loads above the balance point are assumed to contribute to crack control.

Comparison with working stress method

As mentioned previously, the ETL 1110-2-312 SD criterion was developed to yield designs equivalent to the working stress method. Of course, one should not expect a perfect correlation between the ETL and working stress methods. EM 1110-1-2101 presents criteria for working stress design. It calls for an allowable stress of $0.35 f_c'$ in the extreme fiber in compression for flexural members. For Grade 60 reinforcement, the tension stress f_s shall not exceed 20 ksi. At the time of publication of EM 1110-1-2101, ACI 318 allowed a compressive stress of $0.45 f_c'$ in the extreme fiber of concrete. For reinforcement in tension, a stress of 20,000 psi were allowed. In compression, the allowable stresses in reinforcement were equal to 20,000 psi and 24,000 psi for Grade 40 and Grade 60 reinforcement, respectively.

The approach given by Everard (1969) was used to determine the allowable service load capacities (working stress interaction diagram) for the section in Figure 1. Figures 3 and 4 compare the ACI 350R-89 (ACI Sanitary) criteria with the working stress method for $f_c = 0.35 f_c'$, $f_c = 0.45 f_c'$, and $f_s = 24$ ksi. The difference in Figures 3 and 4 is the value of ϕ being equal to 0.7 and 0.9, respectively. This reflects the effect of ϕ. Although curves for f_y equal to 40, 48, and 60 ksi are presented in each figure, only $f_y = 60$ ksi is appropriate for $f_s = 24$ ksi. A load factor of 1.9 was used.

The study of Figures 3 and 4 indicates that the ACI Sanitary curve for $f_y = 60$ ksi is within the $f_c = 0.45 f_c'$ curve at nearly all

locations when the appropriate ϕ value is considered. A similar comparison for the ETL 1110-2-312 criteria is made using Figures 5 and 6.

Conclusions

The current trend is toward the elimination of Grade 40 reinforcement in practice. Grade 40 steel in bar sizes No. 7 through No. 11 may not be available now in many locations.

The two most important parameters affecting the computation of section strength are ε_{cmax} and f_y. A comparison of the nominal strengths (neglecting effects of load and strength-reduction factors) as computed using the ETL 1110-2-312 and ACI 318 criteria indicates that the ETL 1110-2-312 criteria are considerably more conservative than the ACI 318 in most of the region above the balance point. This study indicates that the ACI 350R-89 durability coefficients used in combination with a single load factor of 1.7 is appropriate for Grade 60 reinforcement.

Comparisons with the working stress design criteria indicate that both the ETL 1110-2-312 and the ACI 350R-89 SD criteria are generally within the ACI working stress criterion in all regions, but outside the EM 1110-2-2101 working stress criterion in the region just above the balance point. Although not discussed above, analyses performed on structures subjected to flexure and axial compression (i.e., box culverts and powerhouse intake piers) indicate only a 6 percent difference in the results of using the strength design (letting $H_f = 1.3$) and the working stress methods in the region just above the balance point. It is obvious that (even for SD criteria developed to be equivalent to working stress criteria) the SD criteria do not perfectly match the working stress criteria. However, the match is generally good from the point of zero moment up to the balance point and at zero axial load.

References

ACI Committee 439. 1989 (Jan/Feb). "Steel Reinforcement-Physical Properties and US Availability," *American Concrete Institute Materials Journal*, American Concrete Institute, Detroit, MI.

ACI Committee 318. 1989 (Nov). "Building Code Requirements for Reinforced Concrete and Commentary," ACI 318-89, American Concrete Institute, Detroit, MI.

ACI Committee 350. 1990 (Jun). "Environmental Engineering Concrete Structures," ACI 350R-89, American Concrete Institute, Detroit, MI.

American Society for Testing and Materials. 1987. "Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement," Designation: A 615, Philadelphia, PA.

Everard, N. J. 1969. "Ultimate Strength Design - Serviceability Investigation, a Unified Method for Reinforced Concrete Bridge Design," Paper SP 23-22, American Concrete Institute, Detroit, MI.

Hamby, C. C., and Price, W. A. 1984. "User's Guide for Concrete Strength Investigation and Design," Instruction Report K-84-9, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Headquarters, Department of the Army. 1963 (Nov). "Working Stresses for Structural Design," Engineer Manual EM 1110-1-2101, Washington, DC.

Headquarters, Department of the Army. 1971 (May). "Details of Reinforcement - Hydraulic Structures," Engineer Manual EM 1110-2-2103, Washington, DC.

Headquarters, Department of the Army. 1981 (Sep). "Strength Design Criteria for Reinforced Concrete Hydraulic Structures," Engineer Technical Letter ETL 1110-2-265,

Washington, DC.Headquarters, Department of the Army. 1988 (Mar). "Strength Design Criteria for Reinforced Concrete Hydraulic Structures," Engineer Technical Letter ETL 1110-2-312, Washington, DC.

Headquarters, Department of the Army. 1990 (Jan). "Strength Design for Reinforced Concrete Hydraulic Structures," Draft Engineer Manual EM 1110-2-XXXX, Washington, DC.

Liu, T. C. 1981 (Jul). "Strength Design of Reinforced Concrete Hydraulic Structures, Report 2, Design Aids for Use in the Design and Analysis of Reinforced Concrete Hydraulic Structures," Technical Report SL-80-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

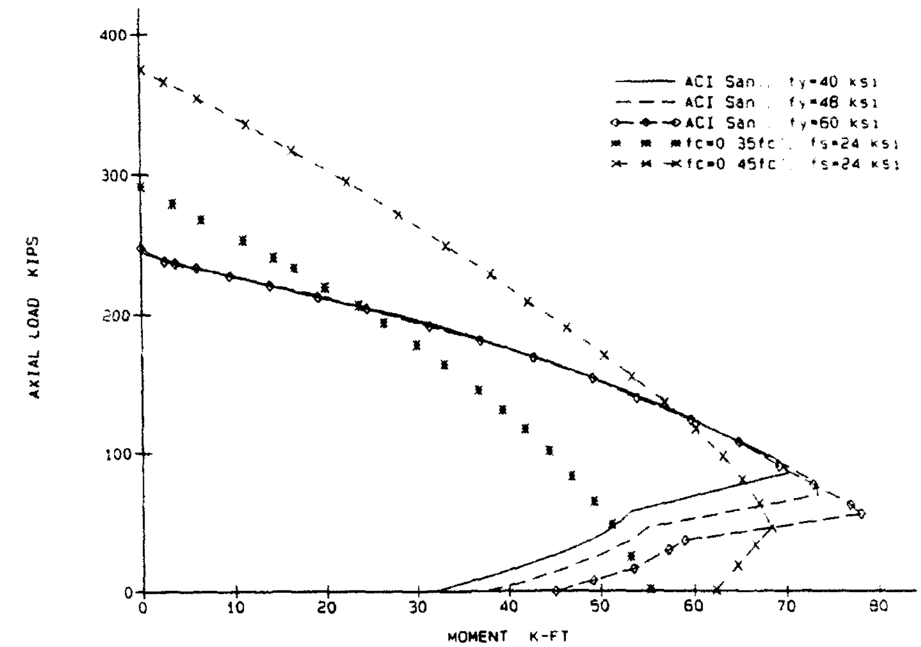

Figure 3. ACI Sanitary versus working stress, $\phi = 0.7$

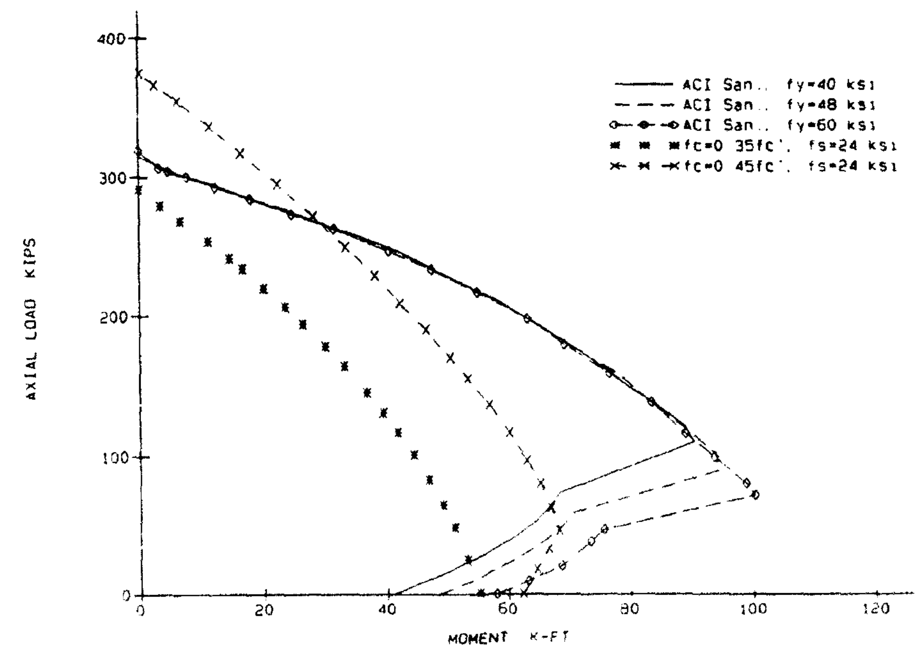

Figure 4. ACI Sanitary versus working stress, $\phi = 0.9$

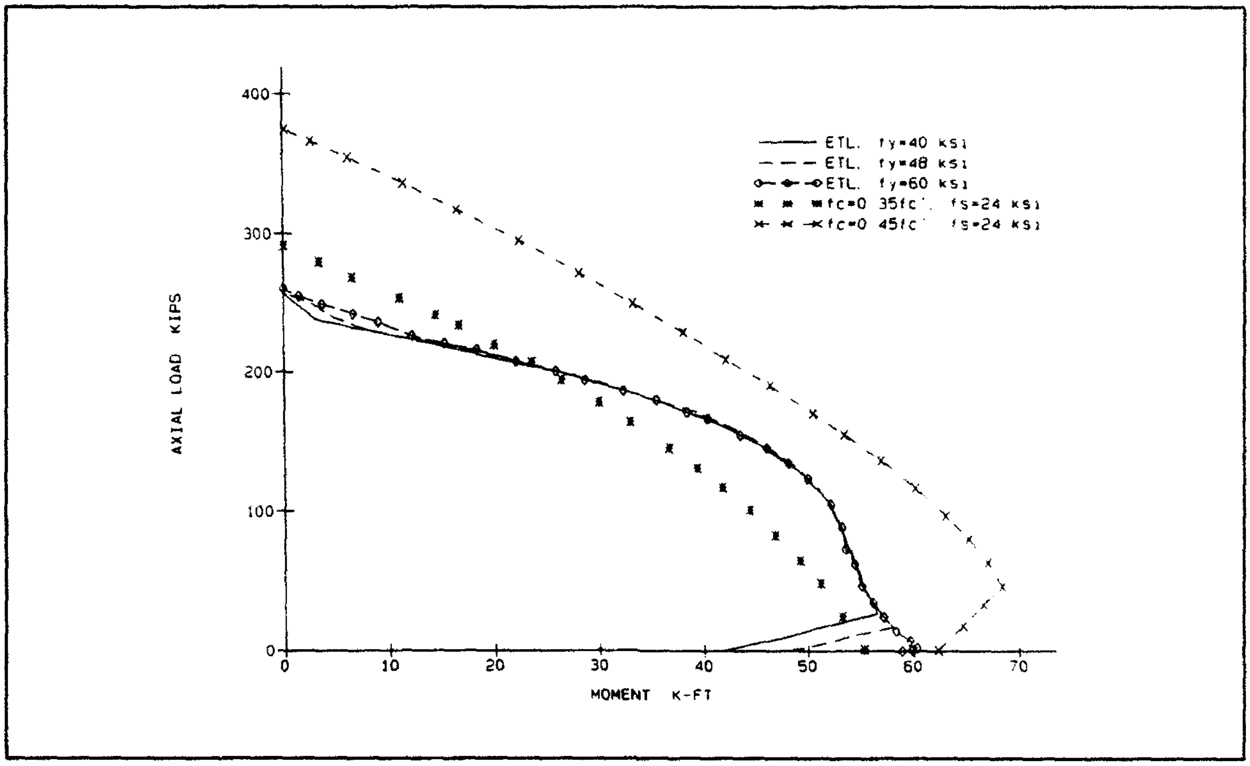

Figure 5. ETL strength versus working stress, $\phi = 0.7$

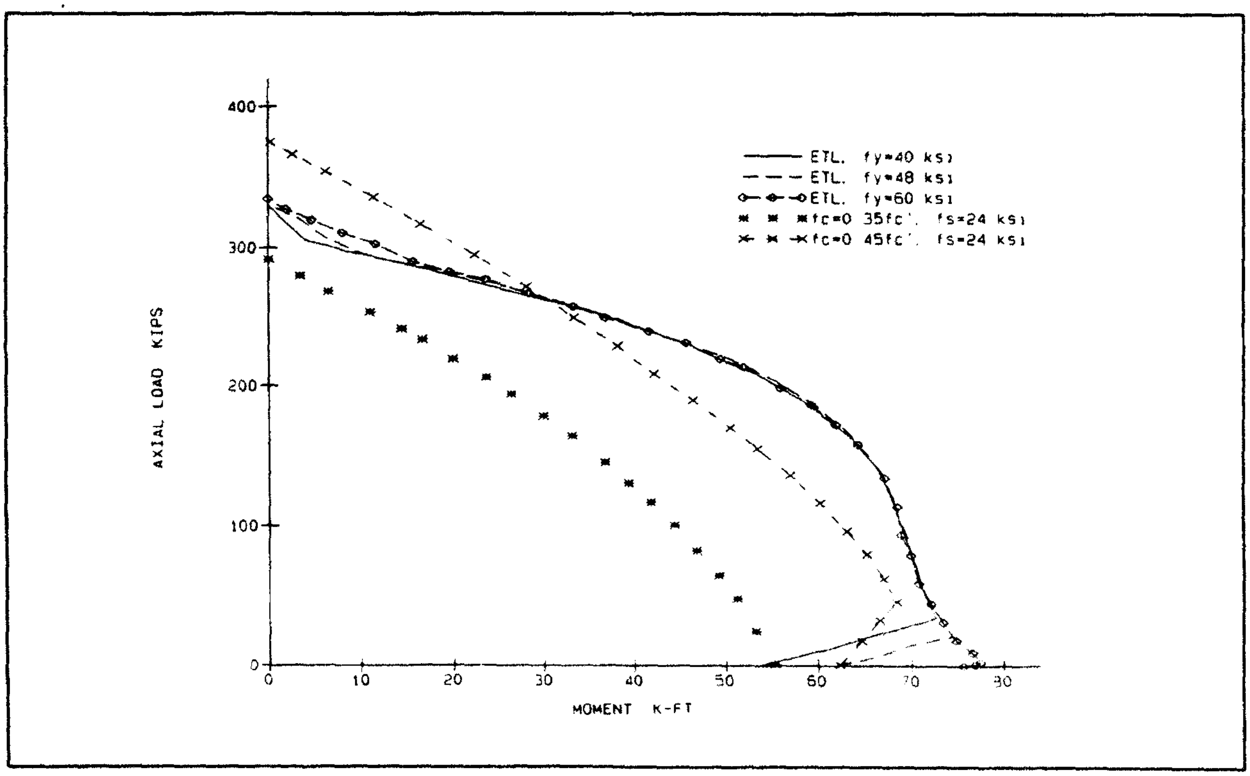

Figure 6. ETL strength versus working stress, $\phi = 0.9$

Condition Assessment and Maintenance Management Decision Support for Navigation Lock Structures

by

David T. McKay[1] *and Anthony M. Kao*[1]

Abstract

The US Army Construction Engineering Research Laboratory (USACERL) has been tasked under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program to develop an automated REMR Management System for Civil Works structures. A REMR Management System combines technologies, condition assessment, economic analyses, and data base management to yield a decision-support tool for planning and budgeting maintenance activities of Civil Works structures. Systems that have been completed and field tested include procedures for miter lock gates, steel sheet pile, and concrete lock wall monoliths in navigation lock structures. Current efforts address other structures and equipment such as training dikes, breakwaters, jetties, sector and tainter gates, filling and emptying valves, and operating machinery. A description of the REMR Management System for navigation lock structures is presented.

Introduction

As the United States moves into the 21st century, we find ourselves facing an aging infrastructure. Bridges and highways crumble beneath us as we try to use funds available for maintenance and repair (M&R) efficiently. One role played by the US Army Corps of Engineers is the design, construction, and maintenance of Civil Works structures. Of the Corps' Civil Works structures for purposes such as flood control, coastal protection, water supply, hydroelectric power generation, and inland waterways navigation, 70 percent are at least 20 years old. The Corps operates nearly 600 locks and dams: 60 percent of these are more than 20 years old, 40 percent are more than 30 years old, and 50 percent will have reached their design life by the year 2000 (McDonald and Campbell 1985). As such, the Corps is shifting the focus of its mission in Civil Works from new construction to repair, maintenance, and rehabilitation of Civil Works.

The Corps' REMR Research Program

The goal of the Corps' REMR Research Program is to identify and develop cost-saving technologies for maintaining and extending the service life of Civil Works structures. Part I of the program was funded with $35 million from 1983 to 1989. REMR II is now underway with another $35 million for 1991 through 1997. Seven problem areas are addressed by REMR research: concrete and steel structures, geotechnical, hydraulics,

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coastal protection, electrical and mechanical, environmental impact, and operations management.

Emerging Technologies in Operations Management

Computerized systems lending decision support to maintenance managers are now receiving much attention. The fundamental elements of a maintenance management system are inventory, objective condition inspection and assessment procedures, a library of maintenance and repair alternatives, and life-cycle cost analyses. The primary goal of such a system is to provide assistance in spending limited resources more wisely. An additional benefit of the system is that it removes many purely subjective elements from the decision-making process.

The planning and budgeting of M&R activities for any facility requires a working knowledge of the following factors: current condition, future condition, operating environment, available M&R alternatives, available funds, and life-cycle cost analyses of various applied M&R policies. Any M&R strategy should include a means to prioritize required work.

One of the developing technologies in operations management focuses on refined concepts in condition assessment. The key word in discussing any M&R-related topic is "condition." Satisfactory descriptions of condition must be developed and adhered to in the attempt to remove subjectivity and ensure objectivity. How good is "good"? What is "fair"? How "bad" is "severe"? A definition of condition that ensures uniformity and is consistently meaningful provides a measure to compare the condition of similar facilities.

With a firm definition of condition in place, it becomes possible to establish guidelines for consistent condition inspection procedures. Uniform inspection procedures yield results, or data, that are repeatable. Repeatable results can then produce objective interpretations of condition.

Quantifying Condition: The Condition Index

The Condition Index (CI) is a numerical indicator of facility condition and function level. The CI is a number between 0 and 100, with 0 representing complete failure and 100 the as-built condition. As a numeric quantity, the CI is certainly objective and obviously can be stored in a computer. Because it is a number, it can also be used in mathematical expressions. By providing a quantitative and consistent scale for describing condition, the CI allows the condition of facilities to be compared and monitored over time.

Table 1 shows the REMR Condition Index Scale. Common terms are defined to constitute a basis for the discussion of condition. The scale is first divided into three general zones, where each zone is indicative of the immediate M&R attention required by the facility. Facilities falling into zone 3 (CI less than 40) require immediate M&R action because the facility's safety is in question. Facilities falling in zone 1 (CI greater than 69) are generally in good repair and no immediate action outside of routine maintenance is called for. Zone 2 facilities (CI from 40 to 69) require judicious planning and budgeting of M&R activities.

The REMR Condition Index Scale is the same for all facilities. The rating process for each facility is different. Each rating process is designed to produce condition indices conforming to condition and function levels set forth in the REMR Condition Index Scale. Generally the more complicated a facility's M&R requirements are, the more detailed the condition rating process will be.

Condition Index Inspection Procedures and Algorithms

The inspection procedures and CI algorithms are developed with the assistance of experts. In most cases these people are Corps personnel who are responsible for the M&R of the given facility. Subcomponents of the facility are determined and possible defects, flaws, or distresses for each are identified. In the simplest

Table 1
The REMR Condition Index Scale

Zone	Condition Index	Condition Description	Recommended Action
1	85 to 100	*Excellent*: No noticeable defects. Some aging or wear may be visible.	Immediate action is not required.
	70 to 84	*Very Good*: Only minor deterioration or defects are evident.	
2	55 to 69	*Good*: Some deterioration or defects are evident, but function is not significantly affected.	Economic analysis of repair alternatives is recommended to determine appropriate action.
	40 to 54	*Fair*: Moderate deterioration. Function is still adequate.	
3	25 to 39	*Poor*: Serious deterioration in at least some portions of the structure. Function is inadequate.	Detailed evaluation is required to determine the need for repair, rehabilitation, or reconstruction. Safety evaluation is recommended.
	10 to 24	*Very Poor*: Extensive deterioration. Barely functional.	
	0 to 9	*Failed*: No longer functions. General failure or complete failure of a major structural component.	

instances, point values are assigned to each distress, with the magnitude of the point value reflecting the distress severity. These points, or Deduct Values, are then subtracted from 100 to yield a CI for the subcomponent. More complicated equations have also been developed that produce CI's with field data as input.

The field observations and measurements used are nondestructive, direct, and simple—usually visual inspection. These observations are related to the physical condition and functionality of the facility. To get as pure a picture of condition as possible, effort is made to avoid introducing age as a parameter affecting the CI. In cases where the safety of the structure is in question, an expanded investigation, including engineering evaluations, should be initiated.

The measurements must be easily repeatable. It is desirable to be able to perform the measurements on in-service facilities with a minimum of downtime. The CI's produced must conform to the REMR Condition Index Scale, and certainly must be meaningful to those responsible for managing the facility.

The Usefulness of the Condition Index

The CI captures a "snapshot" of current condition. Immediately this affords a means to compare the condition of like facilities at both the project (local) level and the network (global) level. For the Corps this equates to comparing facilities at local sites, within Districts, or even within and between Divisions.

With the collection of data over time, it may become possible to predict a facility's condition level. Figure 1 shows a hypothetical curve of CI versus time. Once a minimum allowable condition level has been established, various "what if?" games can be played with the CI. Through life-cycle cost analyses one could determine whether it is economically wiser to perform a full rehabilitation (Policy 1), or a series of repetitive minor fixes (Policy 2), or do nothing at all.

Demonstration: a REMR Management System for Navigation Locks

A REMR Management System for navigation locks is near completion. Completed component systems address concrete lock walls, steel sheet piles, and miter lock gates. A CI is not generated for the entire structure as a whole, but is produced for the walls and gates separately. The condition rating processes for miter lock gates, steel sheet piles, and concrete lock walls are briefly described in the following paragraphs. For a complete description of these systems, see McKay and Kao (1990); Greimann and Stecker (1990);

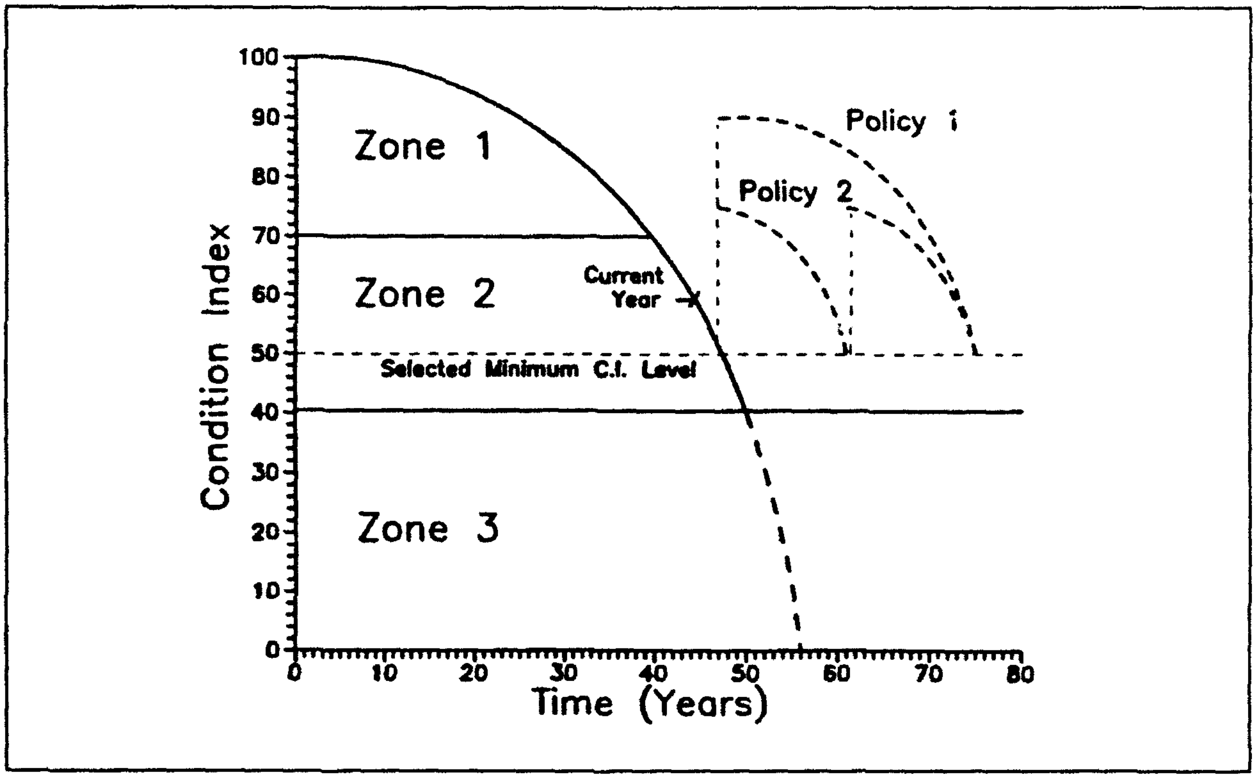

Figure 1. Example application for comparing different maintenance policies

and Greimann, Stecker, and Rens (1990) in References.

The component systems completed so far have been field tested with a high degree of success. The systems have been tested or demonstrated in the US Army Engineer Districts of Rock Island, Nashville, and Tulsa. The inspections proceeded smoothly and without difficulty. The CI's produced have been deemed to accurately reflect the current conditions and functional levels. The same crew performs the inspections for each component of a system. A trained crew can complete an entire inspection, including data entry, in 2 days or less.

Condition Rating for Miter Lock Gates

A series of critical measurements to be made on miter lock gates has been identified. For the purpose of discussion, these quantities are referred to as distresses, and are listed in Table 2.

Experts were asked to interpret each of these measurements in terms of the serviceability and safety of the gate, and assign limiting values to each of them. These measurements may or may not represent serious problems at any given time; but when the upper limit is exceeded, there is indeed a need for further engineering investigation. By considering each distress in its own right, as if it were the only measurable distress on the gate, an upper limit was agreed upon. For each distress magnitude X, there is a maximum allowable X_{max}, as determined by expert opinion.

A component CI for each distress is determined by Equation 1.

$$CI_i = 100(0.4)^{(X/X_{max})} \quad (1)$$

Note that for a given distress measurement $X = X_{max}$, the component CI for that distress becomes 40. According to the REMR Condition Index Scale (Table 1), this is the cutoff point where the condition is rated as poor,

Table 2
Distress in Miter Lock Gates

Distress Code	Distress	Brief Description
1	Top anchorage movement	Motion of upper anchorage system during gate operation
2	Elevation change	Vertical displacement of gate during operation
3	Miter offset	Misalignment of bearing blocks at miter point
4	Bearing gaps	Gaps between bearing blocks at quoin and miter
5	Downstream movement	Downstream movement of miter point as head is applied
6	Cracks	Breaks in structural steel
7	Leaks/boils	Water passing through or around gate
8	Dents	Distortion of steel components
9	Noise/vibration	Abnormal noise, vibration, or jumping during gate operation
10	Corrosion	Loss of steel due to interaction with environment

and where more detailed engineering evaluations are warranted.

The gates are tested during opening and closing, at the fully recessed and near miter positions, and under three stages of static head: low pool, 1-ft head, and full head. Distress measurements are made during each of these steps. Thus for many of the distresses listed in Table 2, multiple measurements are made. Rules have been established that determine a final resultant component CI_i for each of the distress categories.

When multiple distresses are present on the gate, a composite CI is calculated. The functional CI for the gate is given by Equation 2.

$$CI = \sum W_i CI_i \qquad (2)$$

W_i represents a normalized weighting factor. Such weighting factors are again determined by expert opinion, and are listed in Table 3.

During the preliminary field test of the rating procedure, it became apparent that, as a distress becomes more severe, its relative importance grows. To account for this, an Adjustment Factor (AF) was introduced to scale the weighting factor w_i, accordingly. The normalized W_i's were then recalculated. The AF is given in Equation 3.

$$AF_i = 1 \qquad (70 \geq CI_i \geq 100)$$

$$AF_i = 8 - 7\frac{CI_i - 40}{30} \qquad (40 \geq CI_i \geq 69) \qquad (3)$$

$$AF_i = 8 \qquad (0 \geq CI_i \geq 39)$$

Rating systems for other types of gates are also being developed. A rating system for sector gates is currently being tested. Plans for development of a rating system for vertical lift gates will be realized within the next 2 years.

Condition Rating for Steel Sheet Pile Structures

Steel sheet pile in navigation locks is generally found in lock walls and mooring cells. The general approach to the rating system for steel sheet pile structures is the same as for miter gates. A series of critical measurements, or distresses, is identified. An upper limit for the magnitude of each distress is agreed upon. A formula for the component CI_i's is designed to produce results conforming to the

Table 3
Unadjusted Weighting Factors for Distresses in Miter Lock Gates

Distress Code	Distress	w_i	Normalized $W_i(\%) = w_i/\Sigma w_i$
1	Top anchorage movement	11	18
2	Elevation change	9	14
3	Miter offset	5	8
4	Bearing gaps	8	13
5	Downstream movement	7	11
6	Cracks	6	10
7	Leaks/boils	3	5
8	Dents	1	2
9	Noise/vibration	7	11
10	Corrosion	5	8

REMR Condition Index Scale (Table 1). Where multiple distresses occur, a composite CI using normalized weighted averages is employed. Where conditions are severe, the component weighting factors are scaled up accordingly. The equations used for condition rating of steel sheet pile are the same as those used for miter lock gates, namely, Equations 1, 2, and 3. The critical distress measurements and the distress weighting factors for steel sheet pile structures are listed in Tables 4 and 5, respectively.

Table 4
Distress in Steel Sheet Pile Structures

Distress Code	Distress	Brief Description
1	Misalignment	Geometric deviation of sheet pile from design alignment
2	Corrosion	Loss of material due to interaction with environment
3	Settlement	Vertical movement of soil behind sheet pile
4	Cavities	Loss of fill material
5	Interlock separation	Openings in steel sheet pile
6	Holes	Openings in steel sheet pile
7	Dents	Openings in steel sheet pile
8	Cracks	Openings in steel sheet pile

Table 5
Unadjusted Weighting Factors for Distresses in Steel Sheet Pile

Distress Code	Distress	w_i	Normalized $W_i(\%) = w_i/\Sigma w_i$
1	Misalignment	8	23
2	Corrosion	5	14
3	Settlement	4	12
4	Cavities	4	12
5	Interlock separation	4	12
6	Holes	3	9
7	Dents	2	6
8	Cracks	4	12

Condition Rating for Concrete Navigation Lock Monoliths

The rating system for concrete in navigation lock monoliths produces a CI for each monolith inspected. The rating process differs somewhat from the procedure for miter lock gates and steel sheet pile. A series of critical measurements is identified, but rather than assign a component CI_i to each, a deduct value (DV) is assigned. The DV is then subtracted from 100 to obtain a CI for the monolith. In case of multiple occurrences of distress, rules are in place to determine a composite CI.

The CI procedure was developed by assigning specific DV's to defects defined in "Guide for Making a Condition Survey of Concrete in Service" (American Concrete Institute 1980). A "very fine" crack category—cracks less than 0.01 in. wide—was added. The procedure assumes that in most cases, but not all, the ability of a monolith containing a crack to transfer shear is reduced in proportion to crack width. The procedure also assumes that a monolith's tendency to overturn is increased with loss of material due to deterioration. The DV's are subtracted from 100 to establish the CI. Primary DV's were determined with the intent of obtaining a CI of zero when deterioration of a concrete monolith caused a critical threat to the safety of that monolith. Nominal DV's were assigned for defects in serviceability. The resulting CI is indicative of those conditions described by the REMR Condition Index Scale (Table 1).

The distress categories to be considered in a condition rating inspection for concrete monoliths are listed in Table 6.

Crack width, location, and configuration are measured. Volume loss and other loss of section are determined. Exposed steel and damaged or missing armor are accounted for. Leaks, stains, and deposits are accounted for. Although alignment problems are not directly addressed by the current version of the system, the inspection procedure calls for any such problems to be reported immediately. Alignment will be addressed in later versions of the program. For a detailed description of the current rating system, see Bullock (1989).

Table 6
Distress Categories in Concrete Nav-Lock Monoliths

Alignment
Cracking
Checking
D-cracking
Pattern
Horizontal
Vertical and transverse
Vertical and longitudinal
Diagonal
Random
Longitudinal floor
Volume Loss
Abrasion
Cavitation
Honeycomb
Popouts
Scaling
Spalling
Disintegration
Steel Deterioration
Corrosion stains
Reinforcing
Prestress
Armor
Leakage and Deposits
Leakage
Deposits

The inspection procedure relies on visual measurements. It is designed to be performed on in-service locks. It has been demonstrated successfully on high-lift and dewatered locks. A small delay in river traffic may be required to tour the lock by boat at high and low pool. Should severe conditions exist within the concrete, a more detailed engineering evaluation is called for.

REMR Management System Software

All developed REMR systems have been original applications written in C, Fortran, or compiled dBase-compatible languages. They run in a Disk Operating System (DOS) environment on Intel 80286-based microcomputers. The programs are menu driven and user friendly. The programs are available for distribution on diskette. For more information contact US Army Construction Engineering Research Laboratory, ATTN: CECER-EM/ David T. McKay, PO Box 9005, Champaign, IL 61826-9005.

Conclusion

The US Army Corps of Engineers has developed a series of condition inspection procedures for Civil Works structures that yield

uniform and repeatable collections of field data. Such uniformity and repeatability, blended with numeric condition indicators, allow the comparison of condition of similar structures on a global basis. The storage of these data allows a more efficient monitoring of facility conditions. Over time, trends can be monitored and predictions of future condition can be made. Various M&R alternatives, based on knowledge of current condition and life cycle costs, can be laid out. The REMR Management System helps managers obtain the best facility condition possible for a given budget level, and it removes many purely subjective elements of the decision-making process.

References

American Concrete Institute. 1980. "Guide for Making a Condition Survey of Concrete in Service," *ACI Manual of Concrete Practice*, Part 1, American Concrete Institute, Detroit, MI.

Bullock, R. E. 1989 (May). "A Rating System for the Concrete in Navigation Lock Monoliths," Technical Report REMR-OM-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Greimann, Lowell, and Stecker, James. 1990. "Maintenance and Repair of Steel Sheet Pile Structures," Technical Report REMR-OM-9, US Army Construction Engineering Research Laboratory, Champaign, IL.

Greimann, Lowell, Stecker, James, and Rens, Kevin. 1990 (Dec). "REMR Management Systems—Navigation Structures: Management Systems for Miter Lock Gates," Technical Report REMR-OM-8, US Army Construction Engineering Research Laboratory, Champaign, IL.

McDonald, James E., and Campbell, Roy L., Sr. 1985 (Apr). "The Condition of Corps of Engineers Civil Works Concrete Structures," Technical Report REMR-CS-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

McKay, David T., and Kao, Anthony M. 1990 (Sep). "Lockwall: A Microcomputer-Based Maintenance and Repair Management System for Concrete Navigation Lock Monoliths," Technical Report REMR-OM-10, US Army Construction Engineering Research Laboratory, Champaign, IL.

Performance of Microprocessor-Based Steel Detector for Reinforced Concrete Structures

by

W. E. McDonald[1] *and A. M. Alexander*[1]

Abstract

This paper describes the performance of the Profometer 3 as a nondestructive instrument for locating reinforcing steel embedded in concrete. Successful evaluations included tests conducted in the laboratory using prototype specimens as well as field structures located at US Army Yuma Proving Grounds near Yuma, AZ. The evaluation of this device was conducted by US Army Engineer Waterways Experiment Station (USAEWES), Structures Laboratory (SL), Concrete Technology Division (CTD), on behalf of Picatinny Arsenal.

Picatinny Arsenal sponsored a military testing program which involved the construction of reinforced concrete structures. Knowledge of the exact position of the reinforcing steel was critical in the overall evaluation process of the test program. Also, a related aspect of the test program involved the development of a device capable of providing the exact position of the reinforcing steel within specified tolerances, and therefore, an independent calibration source was needed to initially locate the bars.

Laboratory tests performed with the Profometer 3 reinforcing steel detection device indicated the capability of the device to meet the specification requirements. Subsequently, successful onsite demonstration of the capabilities resulted in the request for full-scale field evaluation of the structures.

The Profometer 3 satisfactorily located the positions of the reinforcing steel within the specified tolerances and was subsequently accepted as a verification source.

Introduction

The design of reinforced concrete structures specifies the inclusion of a predetermined percentage of embedded steel reinforcing to satisfy tensile and/or flexural strength requirements. The design specifications indicate the location, size, spacing pattern, and depth for the required reinforcement. Instrumentation devices for providing information on the embedded characteristics of the steel in concrete is, therefore, potentially applicable to all such structures. Recently, microprocessors have been incorporated into steel detectors resulting in devices much improved over past systems. Listings of some of the manufacturers, features, and costs of various systems are provided by REMR Technical Note CS-ES-1.9 (McDonald

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1990). For quality control purposes, verification of compliance with design specifications may be required to assure proper positioning of the reinforcing steel throughout construction. Quality control parameters of concern could include depth of cover, size, location, and spacing. Given that reinforcing steel performs a specific function in concrete structures, improper positioning of these members could compromise the integrity of the structure and subsequently result in failure. The availability of detection devices to verify compliance with the design parameters for positioning of the reinforcing steel could prove to be a valuable tool.

In view of the current trend towards rehabilitating rather than replacing existing distressed concrete structures, the need for capabilities in nondestructively obtaining information about embedded reinforcing steel is now more prevalent than before. Before implementation of a rehabilitation program, an evaluation process should be accomplished as a guide for establishing the scope of rehabilitation requirements. Evaluation and/or rehabilitation of existing concrete structures may require coring, grinding, resurfacing, anchor insertion, etc. Prior to conducting such work, it may be necessary to delineate areas free of embedded steel. Often times, as-built construction drawings detailing information for the embedded reinforcing steel may not be available. Therefore, reinforcing steel detection devices could likewise prove advantageous for in situ assessments.

Background and Scope

Picatinny Arsenal, sponsors of a military test program involving reinforced concrete structures, had concerns about the specified position of the reinforcing steel in several of these structures. The nature of this concern was twofold. First, knowledge of the exact position of the reinforcing steel within specified limits (approximately 0.125 in.) represented one of the critical elements in the overall test program. Secondly, a related aspect of the test program involved the critical examination of a detection device being developed under an existing government contract by a private contractor. The detection device was to be capable of locating the exact position of the reinforcing steel within the specified tolerance. Therefore, an independent and qualified nondestructive testing (NDT) organization possessing an accurate locating device for verifying the performance of the detection device developed under this contract was desired. However, it was not necessary that the reference locating device possess all the military specifications of shock, humidity, color, and other requirements necessary for the contract device. It had to, however, serve as an accurate primary standard for locating the steel.

Enlarging the spectrum of and improving the efficiency of NDT and evaluation techniques for determining the physical and material properties of concrete structures represents one facet of research efforts within the Concrete Technology Division (CTD), Structures Laboratory (SL), US Army Engineer Waterways Experiment Station (USAEWES) (Thornton and Alexander 1987). Consequently, CTD was contacted by Picatinny Arsenal to provide assistance in conducting NDT evaluations for the structures of concern. The scope of the requested assistance included: (1) constructing physical models having steel at known positions for providing confirmation of the capabilities of CTD's in-house laboratory detection device to serve as a legitimate standard for locating the steel within the prescribed accuracy, (2) conducting field demonstrations of the performance of the device, (3) performing onsite evaluation of the actual concrete structures with destructive verification of actual steel location, and (4) providing staff knowledgeable in the physics of NDT devices for follow-up assistance as a consultant for communicating with contract representatives and advising the Army in the evaluation of the contract device.

State-of-art of detection devices

There are a variety of commercially available steel-reinforcing detection devices on the market. Generically, such devices are known as "cover" meters which denotes the capability for measuring the thickness of a concrete

cover overlaying the embedded reinforcing steel (Lauer 1991) or pachometers. The technology of steel-reinforcing detection devices has evolved in sophistication over the years to presently include state-of-the-art circuits, recorders, digital readouts, microprocessors, etc. Previous state-of-the art equipment generally used a coil movement with pointer and scale for reading. The new detector devices now offer improved features of speed, low weight, and ease of interpretation, which have removed some of the impediments that may have hindered routine usage of the devices in the past.

The general principle of most detection devices is the transmission of magnetic flux lines into the concrete through a probe mechanism. Upon encountering steel material, an increased amount of the magnetic flux lines traverses the steel rather than the concrete as a short cut. This is detected by a sensoring circuit in the device which senses an increase in the field strength due to a lower resistance path for the magnetic flux lines. For a given size bar of reinforcing steel, a stronger signal will be transmitted through the steel for a thinner cover than for a thicker cover. Also, for a given thickness of cover, a stronger signal will be transmitted as the diameter increases in the reinforcing steel bar. The transmitted signal is then received, processed, and displayed or recorded by a digital readout, audible beep, meter pointer, or a change in pen movement on a chart recorder.

The steel-reinforcing detection device used to conduct this investigation is manufactured under the brand name of Profometer 3 (Figure 1). This device incorporates a microprocessor system which includes such features as the elimination of induced errors in measurements by controlling the time of the beep signal response to correspond with the time of actual location of the reinforcing steel.

Therefore, the signal response is not dependent upon the particular user having different velocities of probe movement (SDS 1988). A variety of probes are attachable to the device to provide various measuring capabilities including determination of the depth, diameter, and positioning of the reinforcing steel. Other features of this device are fast scanning with beep response (for ascertaining quick, general locations), compensation for magnetic aggregates, audible or digitized visual indicators, easy alternation between operating modes for different functions via a toggle switch, compact and lightweight construction (4.4 lb) for easy handling, and operational simplicity which does not require any specialized training or skill for one to become a competent user after a few trial practice sessions.

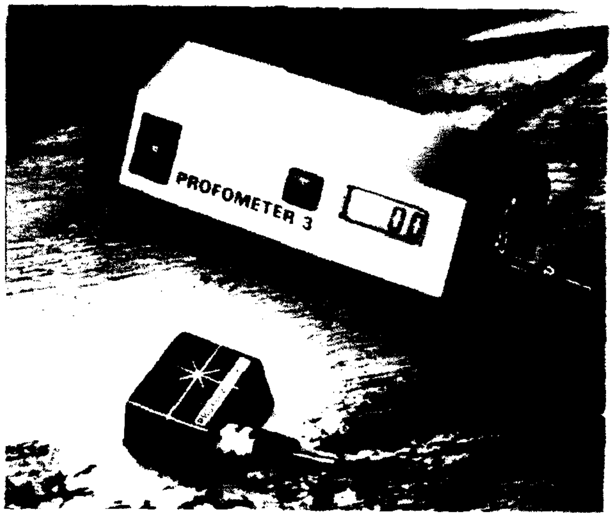

Figure 1. Profometer 3 detection device

Field structures

The field structures evaluated in the full-scale military test program consisted of two typical types of structures, geometrically square and cylindrical. The structures are located at the US Army Yuma Proving Grounds near Yuma, AZ. There are eleven rectangular structures, 5 ft long by 7 ft wide by 15 ft high; and four circular structures, two with 5-ft diam and two with 6-ft diam. All four structures are 15 ft in height.

Design specifications required positioning of #11 vertical bars at 5-3/16-in.-center-to-center spacings and #4 horizontal bars at 12-in.-center-to-center spacings. The center of the #11 bars were located at a depth of 3 in. with the #4 bars directly under the #11's. After

construction of these structures, there was concern that improper positioning of the reinforcing steel in some of the structures may have occurred. As mentioned, the location of the reinforcing steel in the structures represented one of the critical aspects of the overall test program. As a result, any uncertainties had to be verified and resolved prior to eventual testing by the Army.

Investigation

Laboratory evaluation

Two prototype laboratory specimens containing steel with known characteristics were fabricated to simulate the parameters of the field concrete structures. Of particular interest was the simulation of depth, spacing, and diameter of the embedded reinforcing bars similiar to the field structures. Also, there was concern that the presence of miscellaneous metallic materials (i.e. snap form ties and tie wire used to secure positioning of reinforcing bars at intersections) could adversely influence the accuracy of detection capabilities. Therefore, the prototype specimens included embedded metal materials for testing the influence of supplemental, yet, incidental components that had the potential to produce readings that would result in false interpretations of the actual steel embedded characteristics in the concrete.

The two prototype specimens were labeled "A" and "B". Specimen A measured 36 by 6 by 6 in. (l × w × d). Specimen B measured 24 by 24 by 9-1/2 in. (l × w × d). Specimen A contained No. 11 bars placed horizontally and embedded at depth variations of 3-3/4, 3-1/2, 3, and 2-1/2 in. Center-to-center spacings were 7 in. for Specimen B. Both No. 11 and No. 4 bars were placed horizontally and embedded at 3-in. depths and 5-1/2-in. center-to-center spacings. The No. 4 bars were tied on top of the No. 11 bars. Four No. 2 bars were embedded vertically at random locations to simulate the presence of form snap ties. Specimen B was designed to provide identical parameters to the design of the field structures. Figure 2 illustrates prototype laboratory specimens.

In addition to the prototype specimens, scaled down models of the field structures were fabricated and included as part of the laboratory evaluation process.

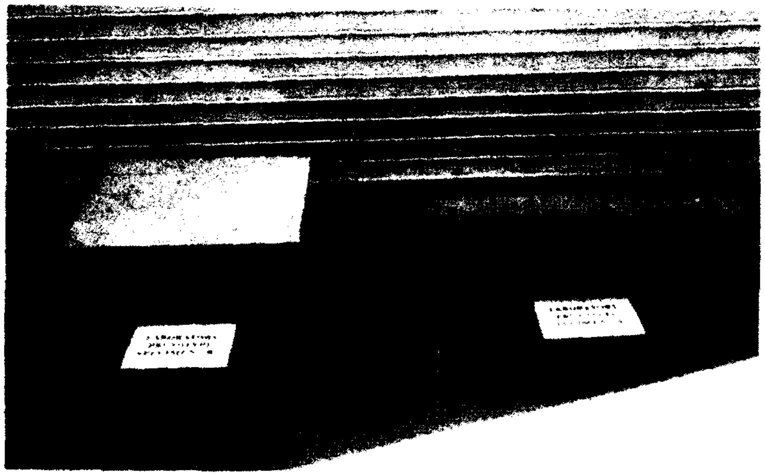

Figure 2. Laboratory prototype specimens

Laboratory results

Initial cursory efforts were made to detect the parameters of the reinforcing steel in the laboratory prototype specimens using detection devices with and without a microprocessor-based system . The device without a microprocessor-based system included coil movement, a needle, and scale for reading and generally performed satisfactorily in most situations. However, it was not capable of resolving the position of the center axes of the reinforcing steel within a specified tolerance for this application. On the other hand, evaluations conducted with the Profometer 3 detection device (a microprocessor-based system) yielded positive results, and therefore, was used throughout the remainder of the laboratory evaluation process.

To evaluate the accuracy and repeatability of the Profometer 3 detection device, reference marks indicating the center axes of the embedded reinforcing steel axes were placed on the top surface of the prototype specimens.

The Profometer 3 was then used to locate the center axes. Repeatability refers to the ability of the detection device to consistently indicate the position of the reinforcing steel axes. Thirty (30) measurements were taken, and the offset distances from the reference marks were measured and recorded. Table 1 gives typical data results. The mean offset from the actual center line in this particular evaluation was 0.081 in. as determined by Equation 1. The standard deviation as calculated by Equation 2 was 0.047 in.

$$\mu = \frac{\sum x}{n}$$

$$\mu = \frac{2.43}{30} \tag{1}$$

$$\mu = 0.081$$

where

μ = mean

x = individual observations

n = number of observations

$$\sigma = \sqrt{\frac{\sum (x-\mu)^2}{n}}$$

$$\sigma = \sqrt{\frac{0.259^2}{30}} \tag{2}$$

$$\sigma = 0.047$$

where % = standard deviation. These results are also typical of other analyses conducted to measure accuracy and repeatability. The lower and upper limits of offset values obtained in Table 1 are 0.00 and 0.2300 in., respectively. The 0.08-in. average offset value is just at the required specified accuracy limit of 0.125 in. since there is a 67-percent probability that each reading will fall below the sum of the mean and the standard deviation. The value would be 0.128 in.

Table 1
Measurement Data on Accuracy and Repeatability

Measurement Observation	Deviation from Center Line, in.
1	0.03
2	0.02
3	0.09
4	0.07
5	0.05
6	0.08
7	0.06
8	0.06
9	0.05
10	0.11
11	0.17
12	0.01
13	0.09
14	0.06
15	0.05
16	0.06
17	0.08
18	0.08
19	0.08
20	0.12
21	0.09
22	0.05
23	0.11
24	0.17
25	0.06
26	0.23
27	0.00
28	0.11
29	0.08
30	0.11

Another phase of the laboratory evaluation included locating the center axes of the reinforcing steel in the scaled-down models of the actual structures. The results from this evaluation proved consistent with the results obtained for the evaluations conducted with the prototype specimens. The reinforcing steel axes were determined within the accuracy constraints even when measurements were repeated days apart. The success of the laboratory evaluations provided ample confidence to proceed with the field demonstration phase of the NDT investigation.

Field demonstrations

The process of performing field demonstrations of the reinforcing steel detection device included determining the location of embedded reinforcing steel in several randomly selected structures. Some of these structures suffered deterioration to the extent that the reinforcing steel could be observed visibly to verify the accuracy of the locations determined. For the other structures selected, construction drawings were available for verifications. In all instances, the detection device again proved its capability for consistently pinpointing the center axes of the reinforcing steel measured.

Field structural evaluation

The field evaluations involved locating the position of the embedded reinforcing steel in the structures as well as verifying the performance of the reinforcing steel-locating-sensoring device developed under contract. The initial evaluation conducted with the Profometer 3 device involved constructing a grid of the measured position of the reinforcing steel in an area of one of the rectangular structures. The grid was drawn on mylar material taped to the structure. To determine the accuracy of the measurements, the concrete cover was destructively removed from a section of the structure within the grid area. The removal of the cover was accomplished so that the outer portions of the grid lines remained visible and intact. After removing the cover and exposing the reinforcing steel, it could be seen that the grid lines drawn using the Profometer 3 indicated the correct positions of the center axes of the embedded reinforcing steel. The Profometer 3 indicated the center axes of the reinforcing steel with only a few deviations. However, the deviations were still within the limits of accuracy acceptable, ± 0.125 in. The Profometer 3 was then used to map the reinforcing steel positions for the remainder of the structures. Upon review of the performance of the device by Picatinny staff and subsequent destructive verification, it was decided that the Profometer 3 satisfied the desired requirements to function as an independent quality assurance device.

Follow-up structural evaluation

The field performance of the Profometer 3 convinced the Picatinny staff that the device was capable of serving as a reference device for locating the reinforcing steel within the specifications desired of the contract detection device. The follow-up phase of this investigation involved sending into the field a CTD consultant, knowledgeable in the physics of NDT instrumentation and in the area of concrete materials. The role of the CTD consultant was to use his experience as an NDT expert and attempt to satisfy the contract representative that the Profometer would serve as a proper standard for evaluating their prototype device. In addition, he was to evaluate the contract device, advise the Pictinny staff of its performance, and provide secondary verification of the grid lines laid out by the previous operator. In terms of the latter, follow-up evaluations of the structures using the Profometer 3 were performed by the CTD consultant, Picatinny staff, and contract representatives.

Once the grid lines were verified, members of the contract staff were convinced of the Profometer's capability to locate the embedded reinforcing steel within the specifications desired. The contract device was then tested against the reference lines to determine its performance.

Conclusion

The Profometer 3 satisfied the specified requirements for serving as an independent calibration source for locating the steel precisely and permitting a proper evaluation of the performance of the prototype contract detection device. The Profometer was capable of locating the horizontal surface position of #11 reinforcing steel within 1/8 in. when the steel was at a depth of 3 in. with a center-to-center separation of 12 in. The #4 bars, at a depth of 3-3/4 in. with a center-to-center spacing of 12 in., were also located horizontally within 1/8 in. The various features of low weight, battery power (days of operation without replacing batteries), digital readout (accurate pinpointing of steel) and auditory response

(for quick overview and low lighting), and ease of operation make for an attractive device for use and should promote its routine use for diagnosing concrete structures. A disadvantage is that the microprocessor-based systems are more costly than previous systems. The device was about three times the price of the nonmicroprocessor-based systems. However, they are likely to be used on jobs more frequently than in the past due to the various features noted which will increase the benefit-to-cost ratio of the device.

Acknowledgements

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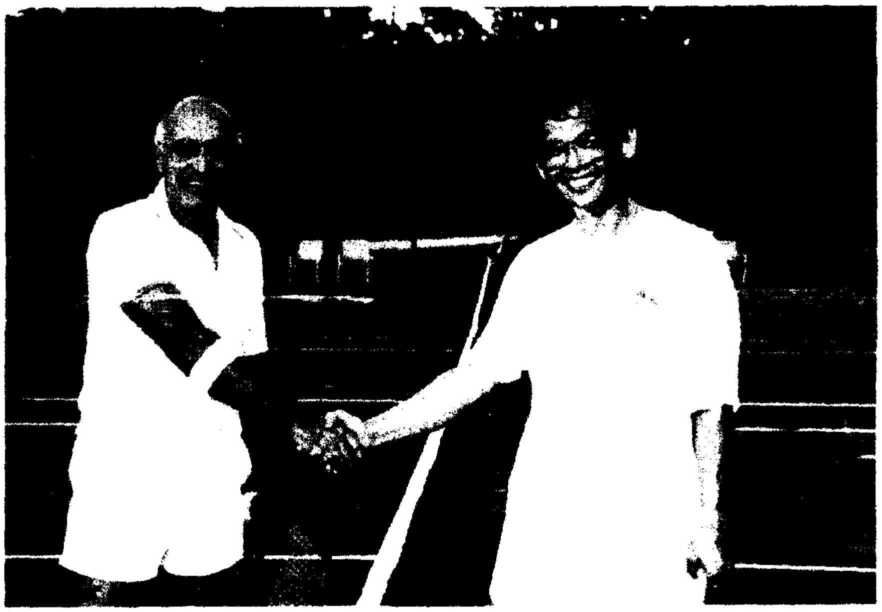
References

Lauer, K. R. 1991. "Handbook on Nondestructive Testing of Concrete," *Magnetic/ Electrical Methods*, Ch. 9, Malhotra and Carino, CRC Press.

McDonald, W. E. 1990. "Detecting Steel Embedded in Concrete," REMR Technical Note CS-ES-1.9, Supplement 4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

SDS Company. 1988 (Feb). Brochure No. 880745E, Paso Robles, CA.

Thornton, H. T., and Alexander, A. M. 1987 (Dec). "Development of Nondestructive Testing Systems for In-Situ Evaluation of Concrete Structures," Technical Report REMR-CS-10, US Army Engineer Waterways Experiment Station, Vicksburg, MS.



Strength Development of Concrete Cured at Low Temperature

by

Charles J. Korhonen, PE,[1] *Edel R. Cortez,*[1] *and Brian A. Charest*[1]

Abstract

Long winters, emergency repairs, or simply a tight schedule may force an engineer to look for ways to cast concrete for structural applications during cold weather. Under adverse conditions such as these, concreting has most always entailed expensive and time-consuming methods of protecting the fresh concrete from freezing temperatures. A newer approach is the use of chemical admixtures that depress the freezing point of water and allow concrete to gain strength at temperatures that are damaging to normal concrete. This paper discusses a study of a series of chemicals that were tested for their effect on strength gain in concrete cured at various low temperatures. The results show that appreciable strength can be promoted in concrete cured at temperatures below freezing when these chemicals are used.

Introduction

The hydration of portland cement is an exothermic reaction between cement and water; hydration begins immediately upon mixing with water and can continue for years. Thus, during the strength development process, the concrete is being continually warmed by internally generated heat. The extent of this warming depends on how quickly heat is evolved and how quickly it is lost from the concrete to the outside environment.

As in most chemical reactions, the rate of hydration is a function of temperature. At temperatures lower than 20 °C, hydration and its resulting strength gain in concrete decreases. For practical purposes, hydration occurs down to about -5 °C (Carino 1984). Though some hydration still occurs below this temperature, very little water is available to react with the cement due to freezing. Specifically, less than 5 percent of the mix water remains unfrozen in fresh concrete at -5 °C (Mironov 1977). Once ice develops, fresh concrete can lose about one-half of its potential design strength.

During cold weather, the concern, therefore, is to conserve the internally generated heat and to maintain a temperature that will give adequate strength and prevent freezing. If the ambient temperature is not too low, insulating the formwork and exposed surfaces should suffice. At lower temperatures, the

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concrete usually must be heated by an external source. This can be accomplished by heating the forms or by building a heated enclosure. When properly applied, these protective measures can produce good results. However, they can also be expensive.

This paper discusses an alternative approach to winter concreting, one whereby conventional protection is eliminated. In this study, a series of chemicals were evaluated for their effect on strength development in concrete cured at low temperature. The chemicals, termed antifreeze admixtures, were selected based on their potential to depress the freezing point of mix water and speed up cement hydration at temperatures below freezing.

Background

Work at the Cold Regions Research and Engineering Laboratory (CRREL) on evaluating these materials began in 1989, progressing from a literature review of foreign and domestic technology (Korhonen 1990), to a laboratory screening (Phase I) of admixtures (Korhonen, Cortez, and Smith 1991), to their low-temperature evaluation (Phase II) in this paper.

The literature report showed that antifreeze admixtures were used in the early 1950's in the Soviet Union, and numerous antifreeze admixtures had been reported in international literature by the mid-1980's. Domestic literature was quiet on this topic. However, a renewed interest in this field has spurred new work resulting in extremely encouraging results. The laboratory screening test was carried out to verify that these chemicals would perform as described in the literature. Preliminary results showed that some did. This study was conducted to evaluate in detail the low-temperature performance of a few of the most promising admixtures.

Although past tests indicate that antifreeze admixtures can be beneficial, little consideration has been given to these materials in the US. This indifference is changing as future research projects are being planned as part of the Corps of Engineers Construction Productivity Advancement Research (CPAR) Program, a cost-sharing partnership between the Corps and industry. The goal of this particular partnership is to develop a commercial antifreeze admixture that will be competitive with foreign-produced admixtures. Since the Federal Government is the biggest buyer of construction services, CPAR efforts, such as these being planned, are expected to benefit the Government and the US economy.

Test Procedure

Four admixtures were chosen for evaluation of their effect on low-temperature strength gain of concrete. Three admixtures were selected from the screening test mentioned above and one was selected from the literature based on favorable reports.

Materials

Table 1 gives the admixtures used in this investigation. A range of dosages was chosen for mixes 2, 3, and 4 to optimize them for temperatures typical to winter concreting. Mix 5 was chosen based on recommendations given in the literature. The mix water was appropriately adjusted to account for the free water contained in a given admixture. No adjustments were made to the mix design for the solid portion of any admixture.

Table 1
Chemicals Used

Mix No.	Chemical	Percent by Cement Weight
1	Control	None
2a	Sodium nitrate+calcium nitrite	6 + 2
2b	Sodium nitrite+potassium carbonate	6 + 0.06
3a	Sodium nitrite	3
3b	Sodium nitrite	6
3c	Sodium nitrite	9
4a	Calcium nitrite	3
4b	Calcium nitrite	9
5	Urea	6

The basic mix design is shown in Table 2. The cement used was Iron Clad type I portland cement made by the Glens Falls Cement Co., Glens Falls, NY. The coarse aggregate was crushed ledge from Lebanon, NH, classified as a metamorphic amphibolite rock. Its average gradation fit American Society for Testing and Materials (ASTM) size no. 6. It had a bulk specific gravity (ssd) of 2.89 and an absorption of 0.5 percent. The fine aggregate was a natural sand with a bulk specific gravity (ssd) of 2.71, an absorption of 1.1 percent, and a fineness modulus of 2.83. Potable water having a pH of 7.0 was used for the mix water.

Table 2
Mix Design

Ingredient	Kg/m^3
Cement	363
Water	163
W/c	0.45
Fine aggregate	856
Coarse aggregate	1071
Chemical admixture	(Table1)

Mixing, casting, and curing

Mixing took place at room temperature. Each mix was made separately in a 0.1-m^3 rotary mixer. The coarse aggregate and some water were briefly mixed to dampen the aggregate. The mixer was stopped, charged with fine aggregate, and the two materials were mixed until they were well blended. Cement was added with the mixer stopped. Mixing resumed and the rest of the water containing the dissolved admixture was added. The cumulative mixing time was 8 minutes.

Specimen preparation consisted of rodding three layers of concrete into 10- × 20-cm plastic cylinder molds. Plastic caps were sealed over the top of each mold to prevent evaporation.

The specimens were stored in 20, -5, -10 and -20 °C rooms within 45 minutes of the time water was added to the mix. Thermocouples, embedded in the center of an extra sample from each mix and placed in the room air, recorded temperatures at 15-minute intervals.

Test methods

Three samples of each mix were removed from each room for uniaxial compression testing at 7, 28, and 56 days. They were allowed to completely thaw before loading to failure. Testing was not done on frozen samples to avoid strength changes caused by ice. Samples were considered to be thawed when the center of an instrumented sample reached 5 °C. At 56 days, a final set of three samples from each mix was stored at room temperature for an additional 28 days before being compression tested at 84 days. The latter procedure was used to document the recovery of strength upon returning the concrete to room temperature.

Results

Strength

As previously mentioned, four admixtures were selected for strength testing in concrete cured at low temperature. In the following discussions, reference should be made to Figures 1 through 5 for strength results at room temperature, -5, -10, and -20 °C. Each data point in the figures represents the average from three strength tests.

Mix 1. This mix contained no admixture. The 20 °C curing temperature results are designated as the "control" for reference purposes in Figures 2 through 5. Although 20 °C is a benchmark often cited in literature, it is instructive to note that the allowable minimum concrete placement and maintenance temperature under field conditions is 5 °C, according to the American Concrete Institute (ACI) (1988). A dashed line denotes the expected strength of mix 1, as if it were cured at 5 °C (Figure 1). The expected strength is based on guidance given by ACI (1988). Obviously, acceptable strengths for winter concrete could be significantly lower than the reference strengths used in this study. Figure 1 also shows that the rate of strength gain is dramatically reduced at temperatures below freezing. Near -20 °C, hydration virtually stops. Therefore, if concrete must be cast and cured at low temperatures, something must be done for it to achieve strength for structural application.

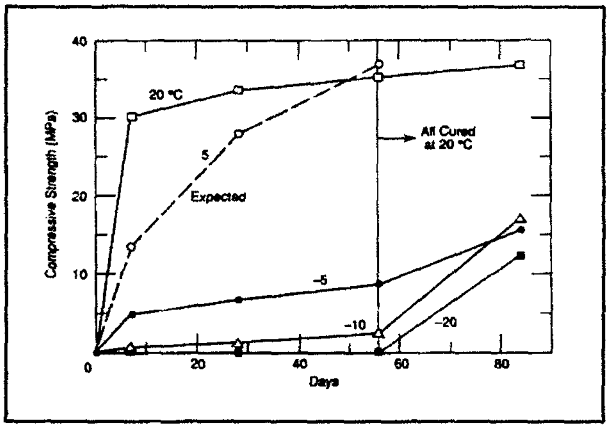

Figure 1. Strength gain of mix 1 concrete at various temperatures. The dashed line, the expected strength of mix 1 cured at 5 °C, is based on ACI guidance (1988)

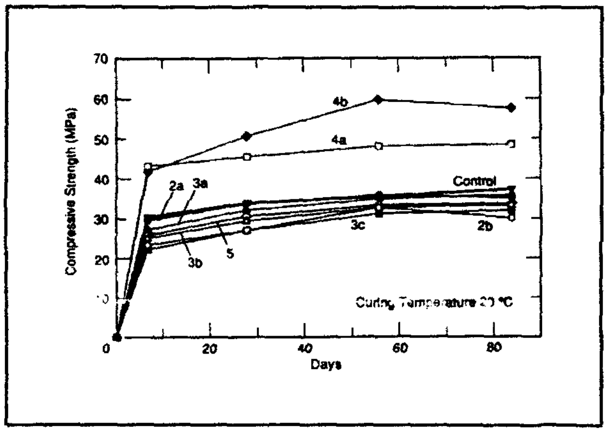

Figure 2. Strength gain at room temperature

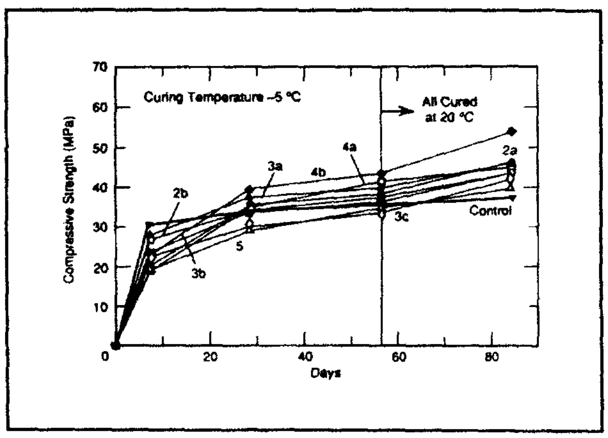

Figure 3. Strength gain at -5 °C

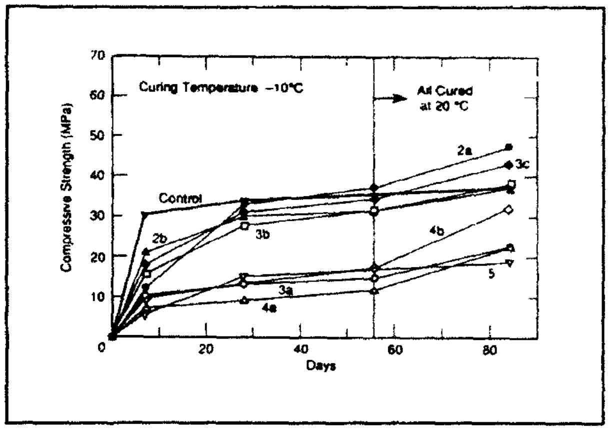

Figure 4. Strength gain at -10 °C

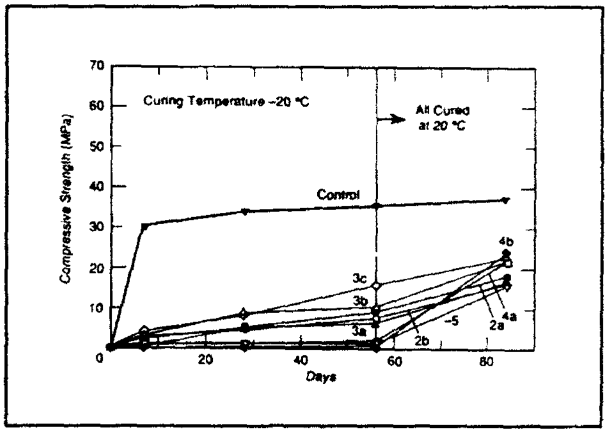

Figure 5. Strength gain at -20 °C

Mix 2a,b. The Phase I tests showed that sodium nitrite, when combined with an accelerator, performed reasonably well at temperatures down to -10 °C. Thus, there existed the potential for strength gain at even lower temperatures and a need for further testing. In this study both mixes again performed quite well, though this time a distinction is drawn between their performances. In general, mix 2a, which contained a calcium nitrite accelerator, performed better than mix 2b, which contained potassium carbonate as the accelerator.

When cured at room temperature, mix 2a matched the control mix in strength age-for-age, while mix 2b showed a 10 to 20 percent loss in strength at all ages. At -5 °C, both mixes were initially weaker than the control

mix, but by day 28 they had both become slightly stronger. Thereafter, mixes 2a and 2b continued to increase in strength relative to the control. At -10 °C, both mixes once again started out weaker than the control. This time mix 2b remained about 10 percent weaker through day 56, at which time the curing temperature was raised to 20 °C, and then recovered full strength at day 84. Mix 2a, meanwhile, continued to increase in strength past that of the control at day 28 as happened at -5 °C . By day 84, mix 2a had become 30 percent stronger than the control. At -20 °C, neither mix developed much strength.

Mix 3a,b,c. This mix was selected to evaluate sodium nitrite by itself as opposed to with an accelerator as in mix 2. Three dosages, one below, one above, and one at the same level as mix 2, were studied. In general, sodium nitrite did not perform as well as it did when an accelerator was used, though it still produced concrete of acceptable strength down to -10 °C. Without an accelerator, sodium nitrite is most useful as a freeze point depressant.

At room temperature, the low dosage mix did not seem to interfere with strength gain, as mix 3a was close to the control mix in strength at all ages. The middle and high dosages, however, showed a tendency toward permanent strength loss. Mixes 3b and 3c produced concrete that was initially weaker than the control mix and that settled in at a 10 to 15 percent loss at a late age.

At -5 °C, mixes 3a and 3b produced strengths that initially were lower than that of the control mix but, past 28 days, became superior to that of the control mix. An interesting result was that the low dosage was better at promoting strength than was the middle dosage. The high dosage produced the weakest concrete in this test series. Nevertheless, all three dosages produced concrete that eventually became stronger than control concrete by day 84.

At -10 °C, the low dosage mix produced the weakest concrete; mix 3a developed only one-half its potential strength at day 56. When the curing temperature was increased to 20 °C, it recovered only up to two-thirds of its potential strength by day 84. The two higher dosages yielded better results, producing concrete that initially was weaker than the control mix but that eventually became stronger. Mix 3c, instead of producing the weakest concrete of the three dosages, as was the case at -5 °C, produced the strongest concrete.

At -20 °C, mixes 3a, 3b, and 3c gained very little strength.

Mix 4a,b. Calcium nitrite was tested in Phase I and performed well at temperatures down to -5 °C. Aside from this, calcium nitrite also has a eutectic temperature around -20 °C and is a key ingredient in a commercial admixture. Thus, it was included for further study. Two dosages were chosen; one below and one above the dosage tested in Phase I. In general, regardless of the dosage used, calcium nitrite seems incapable of promoting strength below -5 °C. It is an excellent accelerator at -5 °C and above.

At room temperature, both dosages produced concrete of exceptional strength. Mix 4b (high dosage) produced the strongest concrete. The same was true at -5 °C where both dosages again produced very strong concrete, with 4b being the best. However, at -10 °C, strengths dropped off significantly, though mix 4b nearly recovered its full potential strength by day 84. At -20 °C, no detectable strength gain occurred; hydration was essentially stopped.

Mix 5. Urea was not tested in Phase I but the literature suggested that a 6-percent dosage rate worked best. In this study, this dosage rate was able to protect concrete down to -5 °C only.

At room temperature, urea reduced concrete strength by about 10 percent at each age compared to the control. At -5 °C, mix 5 initially started out weaker than the control but eventually became stronger by day 84. At -10 °C, mix 5 gained less than half its potential

strength. At -20 °C, hydration was practically stopped, as strength was immeasurable.

Temperature history

Strength gain of concrete is the result of chemical and physical reactions between cement and water. At room temperature, the reaction process is most easily observed as a change in the temperature of curing concrete where the evolution of heat is proportional to the rate of reaction. When first mixed with water, heat is released very rapidly for about 15 minutes. There follows a 1- to 2-hour period of little heat release before a second period of rapid heat release is started. Thereafter, heat release reduces to a low value.

Figure 6 represents a typical heat of hydration curve from each mix cured at temperatures between about 19 and 23 °C. The recordings began approximately 1 hour after water was first added to the cement, and continued well past the second heat-release period. As can be seen, each mix warmed very rapidly during the first 8 to 12 hours before cooling off to a steady value at 20 to 24 hours. In particular, mixes 2 and 4 accelerate the early rate of hydration, mix 3 has minor effect on hydration, and mix 5 seems to delay hydration when comparison is made to the control curve. This supports the strength results in that mixes 2 and 4 provide excellent strengths at an early age, while those of mixes 3 and 5 were somewhat delayed. Mix 5 exhibited the least activity of all mixes at low temperature.

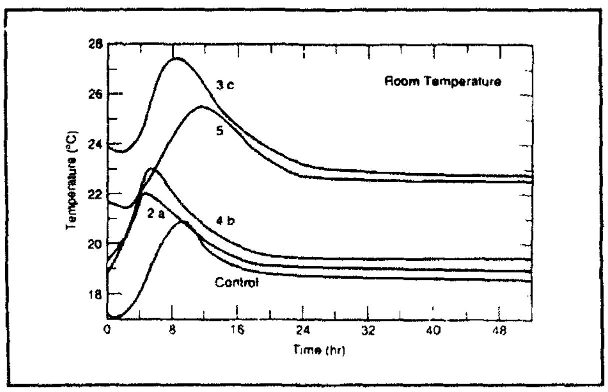

Figure 6. Temperature history of concrete cured at room temperature

Since strength gain of concrete is a function of temperature, the rate decreasing with temperature, it is important to know the temperature of the concrete throughout its curing history. Figure 7 represents a typical cooling curve for concrete cured at each low temperature used in this study. Mix 4c was chosen as the example. As can be seen, the concrete quickly cooled to below freezing in 1 to 3 hours, depending on the curing temperature, reaching room (curing) temperature in as little as 5 to 7 hours. Cooling times were similar for the other mixes.

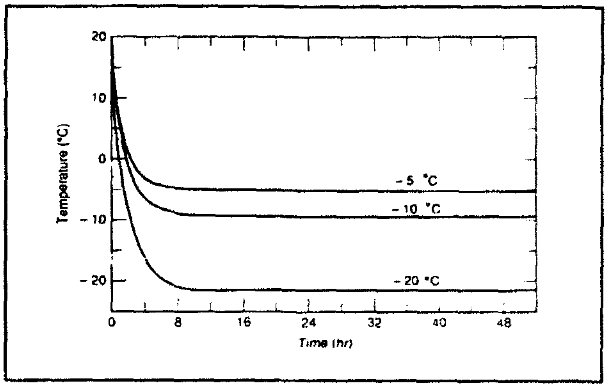

Figure 7. Temperature history of concrete made from mix 4c and cured at low temperatures

From these results, it is obvious that essentially all strength gain in this investigation occurred at the temperature of the curing room. In other words, the curing room temperature can be thought of as the concrete temperature in all cases.

Workability

Workability in Phase I was observed as the relative ease with which various mixes were molded. When compared to the control mix, sodium nitrite had a slight plasticizing effect while calcium nitrite produced somewhat stiffer mixes. In this phase the slump test, as described in ASTM C 143-90 (ASTM 1990), was used as a more quantitative method of indicating workability. Table 3 shows the slumps measured for each mix.

Table 3
Slumps

Mix	Slump (cm)
1	2.5
2a	7.6
2b	8.9
3a	5.1
3b	10.2
3c	12.1
4a	3.2
4b	2.5
5	5.7

As can be seen, all admixtures except for that with the high dosage of calcium nitrite (4b) increased slump. Sodium nitrite increases slump, as discovered in Phase I, but there is little improvement by adding the highest dosage (3c). Slump loss is a concern on the construction site also. For mix 4b, slump loss was quite rapid. Within 20 minutes, this mix became unworkable. For this reason, calcium nitrite at the 9-percent dosage rate is not considered practical for field applications. The other mixes maintained their slump throughout the molding period.

Corrosion potential

Any chemical added to the concrete should not promote corrosion of steel reinforcement. The corrosive potential of each admixture tested in this study was evaluated by immersing a small, sandblasted rebar into an aqueous solution containing the admixture at an equivalent dosage to that used in the concrete mixes. The rebar pieces were kept in the solution for 100 days, and then visually examined for signs of rust. An additional rebar was immersed in a water and cement solution as a reference. Table 4 shows the results. As can be seen, calcium nitrite and sodium nitrite showed no signs of rust. Urea showed more potential to rust than did the reference. More study is needed to verify these performances in concrete.

Table 4
Relative Corrosion

Mix	Observation
1	Light corrosion film
2	No corrosion observed
3	No corrosion observed
4	No corrosion observed
5	Slightly more corrosion than mix 1 (control)

Discussion

Each admixture tested improved the low-temperature strength gain of concrete. Improvement, however, can be a matter of definition. For example, if one requires that low-temperature concrete gain strength at the same rate as that of room-temperature concrete, then none of the admixtures would qualify as an antifreeze admixture even though they clearly can produce concrete of superior strength at late age. They fail primarily because all lack early-age strength to varying degrees.

Perhaps a more realistic criterion for judging antifreeze admixtures might be to require concrete made with an antifreeze admixture and cured at low temperature to gain strength at least as fast as the minimum rate currently permissible for normal concrete in cold weather concreting standards. The dashed line in Figure 1 represents the minimum strength gain permissible for normal concrete used in structural applications (ACI 1988). Accordingly, that translates into strengths that are much easier to meet. For example, antifreeze concrete cured at low temperature should become at least 40, 80, and 100 percent as strong as normal concrete cured at room temperature by 7, 28, and 56 days, respectively.

Table 5 shows the mixes that would qualify as antifreeze admixtures according to the above criteria in the previous paragraph. From this it can be seen that using admixtures to enable concrete to gain adequate strength at temperatures considerably lower than allowed by today's standards is not impossible.

It should also be noted that low-temperature curing can result in a higher late-age strength, even though initial strength gain is slow.

Table 5
Mixes Producing Concrete of Qualifying Strength

Concrete Temperature, °C	Strength Gain at Same Rate as 20 °C Concrete	Strength Gain at Same Rate as 5 °C Concrete
20	4a,4b	all
-5	none	all
-10	none	2a,2b,3b,3c
-20	none	none

If time of cure is not critical, then another criterion by which to judge antifreeze admixtures might be based on late-age strength alone. That is, how cold can concrete get without sacrificing late-age strength? Indications are that several of the -20 °C mixes show promise of eventually recovering their full strength when brought back to room temperature. In one study, concrete made with an antifreeze admixture and cured at low temperature for up to 1-1/2 years gained nearly as much strength as control concrete cured at room temperature (Low Temperature Building Sciences Institute 1979). Though long-term tests are needed to verify this, mix 4b, cured at -10 °C, almost achieved that goal within the time of this study. Applying this criterion would permit concrete to be placed at lower temperature or admixtures to be used in smaller dosages or both.

Regardless of the criteria used to qualify antifreeze admixtures, it is apparent from the results that each mix had its own strengths and weaknesses. Calcium nitrite produced concrete of exceptional strength at temperatures at or above -5 °C. Sodium nitrite, though not as efficient as calcium nitrite in promoting strength, produced significant strengths down to -10 °C. Urea protected concrete down to -5 °C, but reduced the late-age strength of room-temperature concrete.

The performance of these three chemicals may be improved by combining them with other chemicals to offset their weaknesses. For example, the low-temperature range of calcium nitrite may be extended by combining it with a more efficient freezing point depressant. In this manner, its strength-promoting capacity would be useful at even lower temperatures than currently possible when used alone. Sodium nitrite and urea both seem to provide freeze protection but not rapid strength gain at low temperature. In this instance, a low-temperature accelerator may be helpful. Perhaps with these additions, strength gain at -20 °C or lower may be achievable. The literature indicates that strength gain at -25 °C is possible.

Conclusion

Cement hydration will continue at low temperatures provided water is available. The rate of hydration is dependent on the presence of an admixture with the ability to facilitate hydration at those temperatures. These conclusions are supported by the test results in that concrete made with chemical admixtures gained appreciable strength at temperatures significantly below freezing. It is important to realize that the concrete reached these low temperatures within a few hours after water was added to the cement. Thus, strength gain occurred at temperatures damaging to normal, unprotected concrete.

When one considers that concrete can now be "placed and maintained" at least at -10 °C instead of at the more restrictive current limit, the world of winter concreting becomes a bit more friendly. For example, repairs to locks and dams that typically have been done in 0 to -5 °C air temperatures can now be made without heated enclosures. What's more, reduced cracking may well be an added benefit from the lower placing and curing temperatures possible with antifreeze admixtures.

The main drawback to using antifreeze admixtures today is that they are not commercially available. Fortunately, that should be remedied soon as CRREL is working with industry to develop an antifreeze admixture that will meet approved standards.

References

American Concrete Institute. 1988. *Cold Weather Concreting*, ACI 306R-88, *ACI Manual of Concrete Practice*, American Concrete Institute, Detroit, MI.

American Society for Testing and Materials. 1990. "Test Method for Slump of Portland Cement Concrete," Designation C 143-90, *1990 Annual Book of ASTM Standards*, Vol 04.02, Philadelphia, PA.

Carino, N. J. 1984. "The maturity method: Theory and Application," *ASTM Journal of Cement, Concrete, and Aggregates*, Vol 6, No. 2, pp 61-73, Philadelphia, PA.

Korhonen, C. J. 1990. "Antifreeze Admixtures for Cold Regions Concreting, A Literature Review," Special Report 90-32, pp 200-209, USA Cold Regions Research and Engineering Laboratory, Hanover, NH.

Korhonen, C. J., Cortez, E. R., and Smith, C. E., Jr. 1991. "New Admixtures for Cold Weather Concreting," *Proceedings of the Sixth International Specialty Conference*, Cold Regions Engineering. American Society of Civil Engineers, New York, NY.

Low Temperature Building Sciences Institute. 1979. "$NaNO_2$-$NaSO_4$ Combined Additive in Cold Concrete," Draft Translation 717, USA Cold Regions Research and Engineering Laboratory, Hanover, NH.

Mironov, S. A. 1977. "Theory and Methods of Winter Concreting," Draft Translation 636, USA Cold Regions Research and Engineering Laboratory, Hanover, NH.

Underwater Repair of Concrete Using REMR Technical Information

by
Bruce N. Harris, PE[1]

Abstract

Many Corps of Engineers structures are reaching an age at which rehabilitation is necessary. The Corps was aware of the need to develop technical information on materials and construction techniques in the mid-1980's when the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program was started. Today many REMR Bulletins and Technical Reports are available to assist the engineer with development of plans and specifications for rehabilitation of these structures.

The rehabilitation of concrete spalls at the draft tube portals of Gavins Point Dam on the Missouri River used several REMR Bulletins and Technical Reports to provide a durable and cost-effective concrete repair. REMR information gave valuable information on concrete placement technique, installation of anchors, and stay-in-place forms, all of which would be accomplished under water. The availability of the REMR information allowed the underwater repair work to be done quickly and economically since expensive cofferdams for dewatering and loss of power generation revenue due to construction of the cofferdams were not required.

Introduction

Gavins Point Dam is located on the Missouri River near Yankton, South Dakota (Figure 1). It is the smallest of the six Missouri River main stem dams. The powerhouse contains three Kaplan turbines with a generator rating of 33,333 kw each and a plant capacity of 100,000 kw. The average gross head is 45 ft. The power facilities were built in the late 1950's.

Diving inspections had identified concrete spalling at the top of the powerhouse foundation just downstream of the draft tube portals at the interface with the tailrace slab (Figure 2). The inspection also identified concrete spalling of the south retaining wall foundation downstream of the powerhouse at its interface with the tailrace slab (Figure 3). Several void areas existed where water had eroded the shale under the tailrace slab along the length of the retaining wall.

It was decided to fund a concrete repair of the spalled areas when inspections indicated the spalling damage was increasing in area and depth. Another concern was the undermining of the tailrace slab along the south retaining wall.

The plans and specifications would have to address the concrete repair technique

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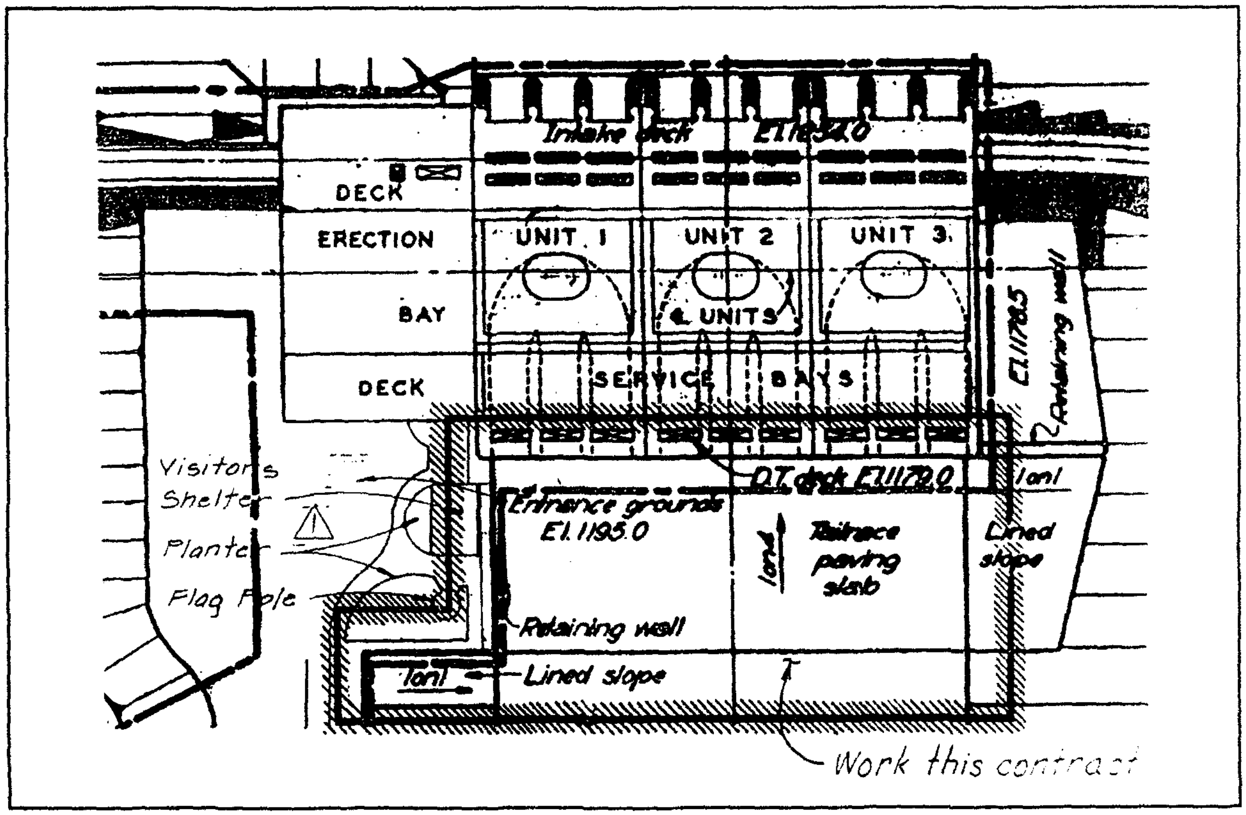


Figure 1. Plan of powerhouse

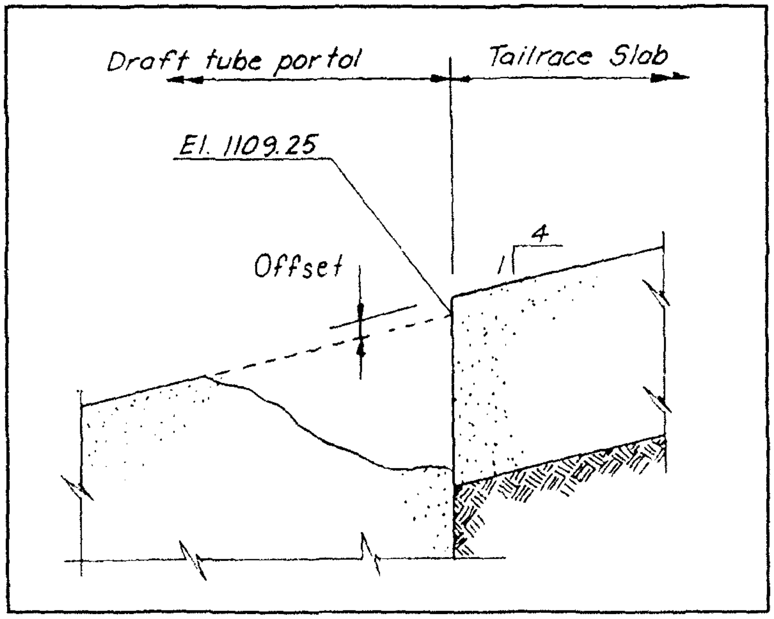


Figure 2. Spall at draft tube portal

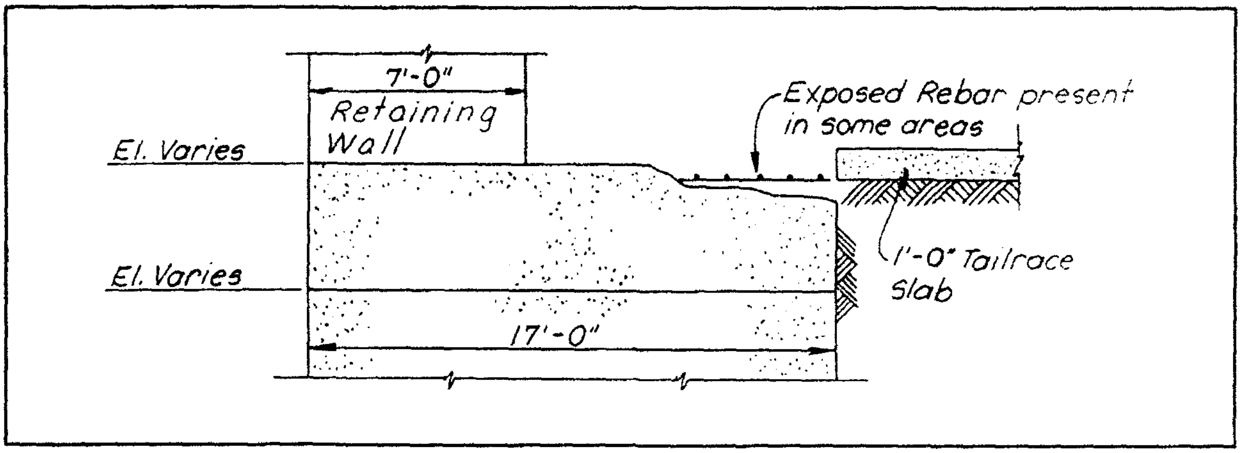


Figure 3. Spall at south retaining wall

complicated by the fact that the repair would be done underwater at a depth of approximately 50 ft.

Reason for spalling

The true reason for the spalling may not be understood completely, but the following description is the author's best scenario. Figure 2 shows the interface between the draft tube portal and the tailrace slab. The powerhouse and tailrace slab are supported on excavated Niobrara chalk and Carlisle shale. The draft tube portal is part of a massive monolithic concrete placement which is part of the powerhouse foundation. The tailrace slab is a 1-ft-thick free-floating slab. The chalk and shale have rebounded over the years since the excavation for the powerhouse. The draft tube portal, being part of the heavy powerhouse foundation, has not rebounded at the same rate as the tailrace slab. The offset between the draft tube portal and slab varies from 1/2-in. to 1-1/2 in. (Figure 2). It is interesting to note that the thinner tailrace slab did not crack and spall; however, it is fully supported by the rebounding shale. In time the water entered the cracks of the concrete causing the top reinforcement to rust, and abrasive effects of rocks and debris caused spalling of the concrete (Figure 3). Over time the spalled areas have increased in depth due to the continuing abrasive effects of rocks and debris over the concrete surfaces during plant operation. The cause of the spalling along the south retaining wall is similar.

Reason for repair

For those who participate in annual or periodic inspections, seeing spalled concrete is not always that much of a concern. However, in this case, the spalled areas have been monitored over the years by diving and/or underwater camera inspections and have recently accelerated in depth and width. The major concern was undermining the tailrace slab since the depth of spalling was getting to be the same as the slab thickness. The tailrace slab along the south retaining wall had several areas where the chalk and shale had been eroded by the water discharged through the draft tube portals, causing a void under the slab.

Proposed repair

The US Army Engineer District, Omaha, Foundation and Materials Section contacted the US Army Engineer Waterways Experiment Station (WES) to discuss the spalling problem and methods of repair. The REMR Bulletins and Technical Reports referenced at the end of this paper were helpful in preparing the construction documents of the selected repair method.

The main goals of repairing the concrete spalled areas and filling the void areas were obvious. However, the construction techniques

and material selections based on that construction were another story. The two basic construction techniques applicable to this repair were constructing it in the dry or under water.

It was felt that the repair done in the dry would be the best. To accomplish the repair in the dry, a cofferdam attached to the powerhouse wall and south retaining wall would be required. The cofferdam would have to be approximately 55 ft high since the water in the tailrace area is approximately 50 ft deep. Even though this method was felt to provide the best repair, the cost of the cofferdam and powerplant outage for this construction technique were too great. Therefore, underwater repair was selected.

The underwater repair consisted of cleaning the spalled concrete surfaces free from drummy rock and loose semidetached or unsound fragments, positioning underwater preplaced aggregate covered by anchored steel or precast concrete units, and injecting grout through injection pipes through the steel plates or precast concrete units to fill all voids in the aggregate (Figures 4 and 5). The voids under the tailrace slab were also filled in the same manner as the spalls. All the work could be done by divers under water.

The steel plate was used as the form for the top surface of the preplaced aggregate concrete repair. The plate was anchored to the sound concrete around and below the spalled area to act as a rigid form, since the grout would be placed under pressure to displace the water in the aggregate voids. It was felt that the concrete repair would bond well with the cleaned existing concrete and would be durable in the abrasive environment without any metal armoring. However, since a form was needed and had to be anchored for the grouting, it was decided to keep the steel plates as an added assurance to prevent future abrasion of the draft tube portal concrete repair.

The precast concrete units were used as a form similar to the steel plates along the south retaining wall (Berger/Abam Engineers 1989). The reason for the precast units instead of the steel plates was that the offset between the retaining wall foundation and the tailrace slab was nearly 1 ft. To make up for the large offset and allow for some future tailrace slab rebound, 15-in.-thick precast concrete units were used (Figure 5). The tailrace slab downstream of the powerhouse was rebounding over ten times more than the area adjacent to the draft tube portal. This could be attributed to the powerhouse weight reducing the tailrace slab rebound near the draft tube portal and permitting more rebound downstream of the powerhouse where its weight has less downward effect.

Another special feature of this repair for which the REMR information gave good insight was in the anchoring of the steel plates and precast concrete units. The selection of anchors was important if the plates were going to stay in place when the powerplant was placed back in operation. Expansion anchors were not used since the hydrodynamic forces on the plates would be vibratory; therefore, adhesive anchors were selected for use.

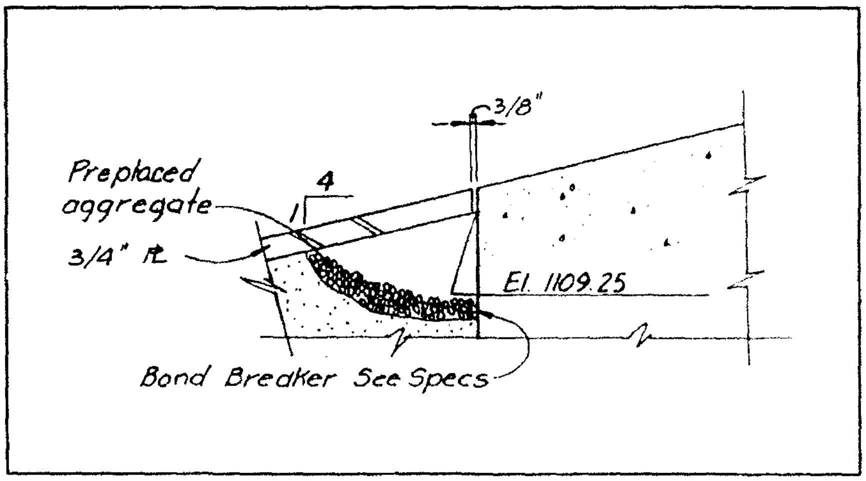


Figure 4. Repaired spall of draft tube portal

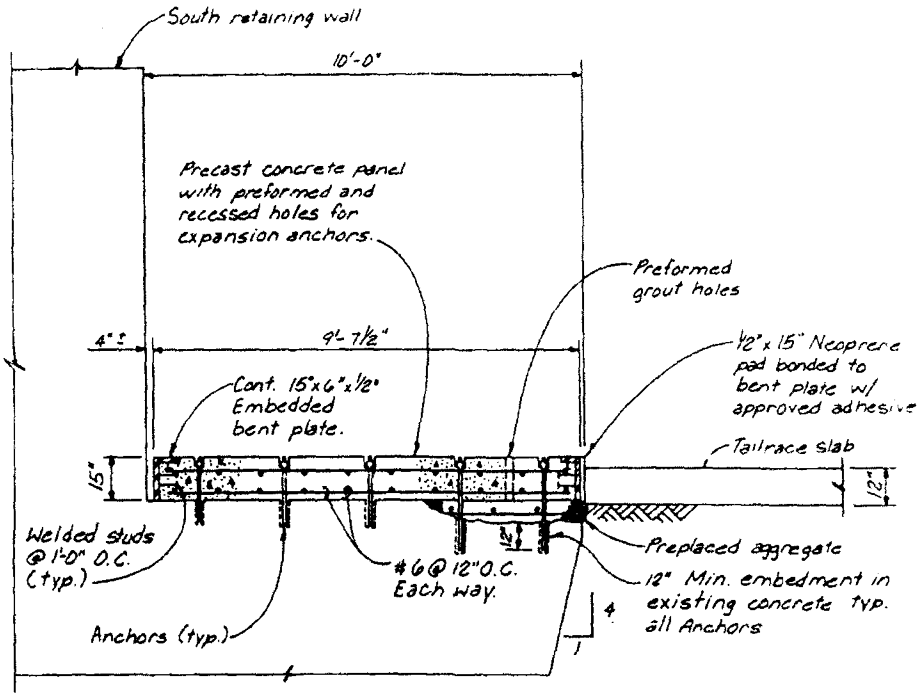
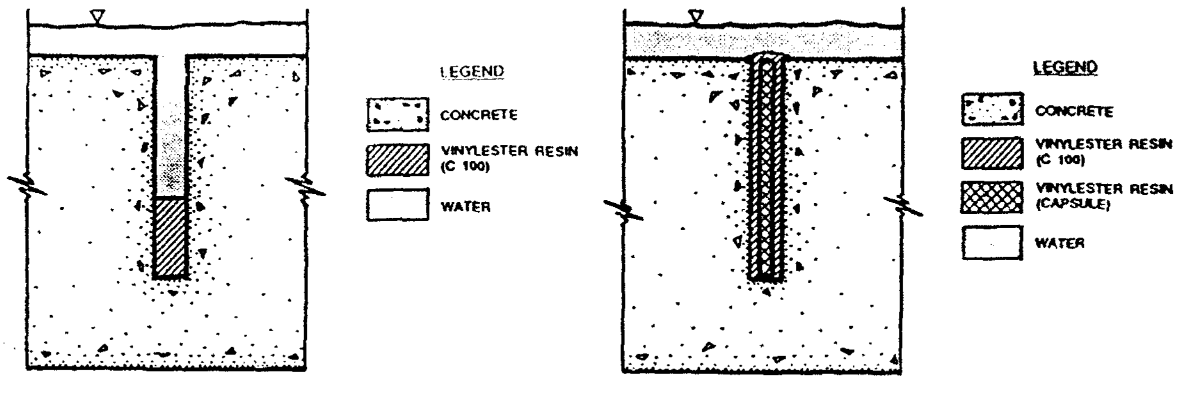


Figure 5. Repaired spall at south retaining wall

REMR information stated that adhesive anchors installed under submerged conditions had an average tensile capacity 75 percent less than anchors installed in the dry (Best and McDonald 1990). The reason for the reduced tensile strength for submerged conditions is the mixing of the resin and water in the drill hole (McDonald 1988). The contractor on this project worked with Hilti, Inc., who in turn had worked with WES to develop a procedure to solve the problem of the water mixing with the resin. The procedure basically uses a doweling adhesive called C-100 in bulk that fills the hole approximately half full. A vinylester resin capsule is inserted into the hole allowing the C-100 to displace the water. Now the two-part capsule can be broken by the threaded anchor rod and mixed without water being present (Figure 6) (McDonald 1990). Tests on tensile strengths using this procedure have yielded nearly the same capacities as those tested after installation in dry conditions.



a. Step 1 b. Step 2

Figure 6. Anchor installation procedure (from McDonald 1990)

Material for repair

The following materials were used in the repair:

Steel Plate	ASTM A-36 (ASTM 1989d)
Precast Concrete	Compressive Strength (28 days) 5,000 psi Reinforcing Bars ASTM A-615, Grade 60 (ASTM 1989a)
Preplaced Aggregate	ASTM C-33 (ASTM 1990)
Fine Aggregates	
Seive Size	Percent by Weight, Passing
No. 8	100
No. 16	95-100
No. 30	55- 80
No. 50	30- 55
No. 100	10- 30
No. 200	0- 10
Coarse Aggregates	
Sieve Size, in.	Cumulative
1-1/2	100
1	90-100
3/4	20-55
1/2	0-10
3/8	0- 1
Cement Grout Portland Cement	ASTM C-150, Type I or II (ASTM 1989c)
Pozzolan	ASTM C-618, Class C or F (ASTM 1989b)
Fluidifier	CRD-C-619 (WES 1985)
Preplaced Aggregate Concrete	Compressive Strength (28 days) 4,000 psi
Maximum Water-Cement Ratio	0.50
Air Content	ASTM C-231 (ASTM 1989e) within 15 minutes after mixing 9.0 ± 1.0 percent
Cement Grout Flow Consistency	18.0 ± 2.0 seconds when tested in accordance with CRD-C-611 (WES 1989)
Adhesive Anchor	1-1/4-in.-diameter 4140 threaded rods set in 1-1/4 by 15-in. HEA capsules and C-100 vinylester adhesive.

Completion of work

Repair work of the type discussed in this paper requires close coordination with regulatory, environmental, and power administrations. These groups are normally affected by the total shutdown of all power units.

In this repair the power units were shut down for a period of 14 consecutive days, during which time the contractor could schedule and accomplish all of the underwater construction activities. No time extensions to the shutdown period were granted due to adverse weather conditions.

If the contractor failed to complete the underwater construction activities within the 14-day power unit shutdown, resulting in an extension of the shutdown period, the contractor would have to pay the Government $36,000 for each additional day as liquidated damages. Conversely, the contractor was given an incentive for completing the underwater construction activities in a satisfactory manner in less than 14 days. The contractor could receive $18,000 per day (1/4-day increments) up to a maximum of 4 days for completing the project earlier than the 14 days.

The contractor was very conscientious and completed the project 3-1/2 days early. To accomplish this the contractor went through every procedural step with his crews on land so that the only variable would be the underwater aspects. Cement grout was prepackaged to assure proper proportioning. The Contractor had several diving crews that allowed him to work 24 hours a day. The Corps monitored the material quantities to assure all spalls and voids were being filled. An independent diving company was used to assure work was completed satisfactorily.

Conclusion

Materials and procedures have been developed to repair concrete under water with good results. Repair of concrete under water is very attractive given the cost of cofferdams that allow the repair to be done in the dry. The installation of adhesive anchors under submerged conditions must follow a procedure where water is removed from the hole so mixing of water and resin will not occur.

Acknowledgements

Special thanks to the Contractor, Atlantic Diving and Marine of Wilmington, NC, for the conscientious preparations and commitment in providing a quality concrete repair. We also thank Mr. James McDonald, WES, for his consultation on concrete repair and his work with the REMR program, and the Gavins Point Project Office for around-the-clock construction management during the construction contract.

References

American Society for Testing and Materials. 1989a. "Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement," Designation A-615, *Book of ASTM Standards*, Part 01.04, Philadelphia, PA.

________. 1989b. "Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement," Designation C-618, *Book of ASTM Standards*, Part 04.02, Philadelphia, PA.

________. 1989c. "Specification for Portland Cement," Designation C-150, *Book of ASTM Standards*, Parts 04.01, 04.02, Philadelphia, PA.

________. 1989d. "Specification for Structural Steel," Designation A-36, *Book of ASTM Standards*, Part 01.04, Philadelphia, PA.

________. 1989e. "Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method," Designation C-231, *Book of ASTM Standards*, Part 04.02, Philadelphia, PA.

________. 1990. "Specification for Concrete Aggregates," Designation C-33, *Book of ASTM Standards*, Part 04.02, Philadelphia, PA.

Berger/Abam Engineers. 1989 (Dec). "Concepts for Installation of the Precast Concrete Stay-In-Place Forming System for Lock Wall Rehabilitation in an Operational Lock," Technical Report REMR-CS-28, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Best, J. Floyd, and McDonald, James E. 1990 (Jan). "Evaluation of Polyester Resin, Epoxy and Cement Grouts for Embedding Reinforcing Steel Bars in Hardened Concrete," Technical Report REMR-CS-23, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

McDonald, James E. 1988 (Jul). "Evaluation of Vinylester Resin for Anchor Embedment in Concrete," *REMR Bulletin*, Vol 5, No. 2, pp 1-5.

McDonald, James E. 1990 (Oct). "Anchor Embedment in Hardened Concrete Under Submerged Conditions," Technical Report REMR-CS-33, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Waterways Experiment Station. 1985. "Grout Fluidifier for Preplaced-Aggregate Concrete," CRD-C-619, *Handbook for Concrete and Cement*, Vicksburg, MS.

________. 1989. "Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method)," CRD-C-611, *Handbook for Concrete and Cement*, Vicksburg, MS.

CORPS/CASE/GCASE Program Demos

by
H. Wayne Jones[1]

The Conversationally Oriented Real-Time Programming System (CORPS) library contains approximately 200 design/analysis programs in hydraulics, soils, structures, and other areas of civil engineering. The CORPS, which uses existing field-developed application programs, is operated by the Scientific and Engineering Applications Center, Computer-Aided Engineering Division, Information Technology Laboratory, US Army Engineer Waterways Experiment Station, under the direction of the Engineering and Construction Directorate, Headquarters, US Army Corps of Engineers.

The mission of CORPS is to provide the noncomputer-oriented engineering with a set of proven engineering application programs, which are available Corps-wide on a variety of computer hardware. The CORPS is available on Division Honeywell, District Harris, the CEAP computers, IBM PC/XT/AT and compatible microcomputers, Intergraph CADD workstations, and the commercial vendor Power Computing.

The CORPS library programs have uniform execution, input, and output procedures. They are well documented and technically supported. The CORPS has proven to be a valuable asset to Corps engineers in their design and analysis functions. All CASE and GCASE programs become part of the CORPS library when completed.

Demos will be available for the following CASE/GCASE programs:

3DSTAB	CBEAR
CTWALL	CSETT
ARCHDAM	CSLIDE
CPGA	CSANDSET
CASM	UTEXAS2
CWALSHT	FE/SEEPAGE
CFRAME	

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CASE Arch Dam Workstation Workshop

by

H. Wayne Jones[1] *and William Wigner*[2]

Abstract

The Jacksonville District is currently completing the design of Portugues Dam, the first arch dam to be designed by the Corps of Engineers. In order to capture the expertise developed during this design effort, the CASE Special Task Group on Arch Dams was formed to assimilate the knowledge gained and to develop a set of tools that can be easily used by other Corps offices. It was decided that this could be best accomplished by developing an Arch Dam Workstation. To do this an integrated software system is being developed which automates (1) the initial geometry layout and static design of the arch dam using the trail-load method, (2) the final static and dynamic analysis of the dam, reservoir, and the foundation system using the finite element method, and (3) the graphical display of the results. This workstation is a new approach to engineering design and analysis by the CASE project and will only be used under a select set of circumstances. This workshop follows the use of the workstation to perform layout and analysis of a simple arch dam.

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[2] Civil Engineer, US Army Engineer District, Jacksonville; Jacksonville, FL.

Computer-Aided Structural Modeling (CASM)

by
Mike Pace[1]

Abstract

The Computer-Aided Structural Modeling (CASM) computer program is designed to aid the structural engineer in the preliminary design and evaluation of structural building systems by the use of 3-D interactive graphics. CASM allows the structural engineer to quickly evaluate various framing alternatives in order to make more informed decisions in the initial structural evaluation process. The program was developed by the Information Technology Laboratory under the Computer-Aided Structural Engineering (CASE) project in conjunction with the Building Systems Task Group.

The demonstration will be partly automated (either using GRASP or Windows macro recorder feature) and partly live. The main functions of the program to be covered (to give the potential user a broad idea of what the program does) are:

- ***Basic Design Criteria.*** *The user can enter information directly or retrieve information from a user definable database. The design criteria include information about the project, regional design information, and site-specific design information.*
- ***Building Geometry.*** *The user can assemble the building shape using 3D primitives (cubes, prisms, spheres, cylinders, etc.) in an easy manner using pull-down menus, icons, and a mouse.*
- ***Dead and Live Loads.*** *The user can select and construct dead and live loads from several user definable menus of building materials and load conditions. These loads can then be applied to any desired area of the building volume.*
- ***Snow and Wind Loads.*** *These loads are automatically calculated in 3D using information from the Basic Design Criteria database. Wind loads are also calculated for components and cladding and open roof structures.*
- ***Structural Layout.*** *The engineer can easily and rapidly experiment with various framing schemes inside the defined building volume. Beams, girders, joists, girts, columns, and walls are some of the structural elements that can be modeled.*
- ***Member Analysis and Preliminary Sizing.*** *The user can apply loads to the building geometry from a list of user-defined load cases. The shear, moment, and deflection of selected members may be calculated for various loading*

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conditions (including pattern loads) and connectivity (including continuous beams). The design of a member is performed using a spreadsheet.

The above functions will be demonstrated by taking a typical building example and progressing through the building system selection.

SAS Software System

by
Dr. Mary Ann Leggett[1]

SAS System is a fully integrated data management tool for analyzing data and generating reports. The software consists of the following separate modules:

- **BASE:** used to store, retrieve, and modify data; compute simple statistics; and create reports. DIF(Lotus) and DBF(dBase) files can be used as input.
- **STAT:** comprehensive statistical package.
- **GRAPH:** high-resolution graphical package capable of producing charts, plots, contour plots, slides, maps, and fonts.
- **ETS:** time series and econometric package.
- **OR:** project management package which includes linear programming and operations research functions.
- **QC:** experimental design and quality control package.
- **IML:** interactive matrix language package.
- **FSP:** full-screen information processing package whichincludes menu building routines for data entry, editing, letter writing, and spreadsheets.
- **AF:** interactive menu building package for applications developers to build customized packages.
- **RTERM:** terminal emulator package.
- **ASSIST:** menu-driven, multifunction interface to SAS's most widely used capabilities. SAS program is automatically built as one chooses from the menus; this file can be viewed, stored, and edited for later use.

PC demonstration will present examples using each of these modules applied to engineering problems with emphasis on structural applications. Handouts for each module will be provided, along with information on availability of the SAS software system through the COE site license.

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Expected Stresses in Dolos Armor Units

by

Tommy L. Bevins[1]

Abstract

As part of the Crescent Dolos Study conducted by the US Army Engineer Waterways Experiment Station, a design guideline is being developed for dolosse. The work in this report estimates the static stress levels and gives recommendations for the maximum size of armor unit that can be used. An engineer can use this information in the preliminary design phase of a breakwater.

Static stress levels for dolosse are calculated using finite element analyses. These stresses are then used to establish the maximum size that can be used for an unreinforced dolos. The maximum, or critical, size for static stresses is when the static stresses exceed an acceptable level. This level must leave sufficient strength for the dolos to resist the pulsating forces generated by wave action.

The analysis is a pseudo Monte Carlo simulation. Several support conditions and orientations of an individual dolos are used in the calculations. The support conditions were preselected to represent typical support conditions seen on a breakwater and to represent extreme support conditions.

The final result from this work is critical size plots with an associated confidence level. These plots would allow the engineer to estimate the static stresses for the dolos size being used and determine if dolosse are a practical design for this breakwater.

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Airblast Prediction for Enclosed Spaces

by
Timothy Knight[1]

Abstract

Accurate airblast prediction is a particularly difficult problem in enclosed spaces such as rooms and ducts because of the complex interaction of multiple waves. Two computer programs that have been developed to help in this area are the SPIDS and BLASTX.

The Shock Propagation in Ducting Systems (SPIDS) program is a first principles hydrodynamic code that can trace the one-dimensional propagation of airshock throughout a multibranched duct system. The program is applicable to a wide variety of problems from small air pipes to large tunnels. The program can model real gas behavior at high pressures and temperatures and can accurately account for multiple wave interactions within a duct. The program is IBM PC compatible, has interactive pre- and post-processors, and can give a wide variety of graphical outputs to the screen or plotter.

The BLASTX computer program predicts internal airblast effects for a single room or series of interconnected rooms due to either external or internal explosions. The combined effects of the multiple reflected and diffracted shock waves as well as the detonation product gas pressure environment are predicted. The code is IBM PC compatible, uses "Key word" file input, and outputs the pressure history at any selected target point as well as the average pressure history over a grid of target points.

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Nonlinear Analysis Technology

by

Bruce Walton[1]

Abstract

The design of structures subjected to loads from weapons and accidental explosions usually involves a nonlinear dynamic analysis. This paper will discuss nonlinear analysis using the general purpose ADINA computer code. Both geometric and material nonlinearities will be discussed. The paper will focus on the nonlinear analysis of reinforced concrete structures and will illustrate the modeling of tension cracking and compression crushing of concrete and plastic behavior of reinforcing steel. Example problems will be presented that cover both model development and calculation results.

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Design of Training Wall Extension, Harry S. Truman Dam, Missouri – Richard A. Shanks, Structures Section, US Army Engineer District, Kansas City

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A Personal Perspective to the Corps' Modernization of Civil Works Guidance Criteria

by
Thomas J. Mudd, PE[1]

A Personal Perspective

Personal Reminiscences Using Corps Guidance

During my assignment to the St. Louis District, in 1969, very early in my professional career, Richard Armstrong took the initiative to create and teach a graduate level course called CE435 "Design of Hydraulic Structures." I remember that Dick assigned me the task of carrying about 30 copies of all the Corps' structural engineering manuals (EM's) to be used by the students as the "textbook" for the course. From the US Army Engineer Waterways Experiment Station (WES) to St. Louis, that suitcase split at the seams and was never the same again. Of course, all the structural engineers that worked with Dick in St. Louis were expected to sign up for this course. This was a very enlightening semester, as most of us young, but know-it-all, structural engineers had not thoroughly studied the Corps' EM's unless we were obliged to use one during a particular design assignment. I remember some very spirited discussions, both in the class and afterwards at the local watering holes, concerning the state of the guidance contained in these manuals. Although the guidance was mandated for our specialties in designing hydraulic structures such as flood walls, pump stations, gravity dams, conduits, tunnels, and navigation structures, it did not address some of the computerized technology just entering the design environment, such as finite element analysis, nonlinear analysis, soil-structure interaction, pile foundation analysis, and design of cofferdams. Considering the state of the technology, we believed that the Corps' guidance was the best available at that time and allowed safe, conservative designs based on precedent developed from accumulated knowledge and experience. Most of the current industry structural codes, textbooks, and guidance were orientated towards building, highway, and railroad-type work. That was fine for bridges and buildings, but if you needed to design a dam, a tainter gate (named after Captain Taintor), or navigation lock, we had to use the Corps' criteria. The St. Louis District (SLD) hired a Professor, Dr. Joseph E. Bowles, who worked the summer of 1968, and he used his time with us rather productively to gather everything he could about Corps criteria and design procedures. He later used much of the Corps' examples to publish a very popular textbook, *Foundation Analysis and Design*, using and embellishing on this Corps material. Dr. Bowles' book entered our design repertoire as some state-of-the-practice guidance for soil-structure interaction analysis and design. Much of that same Corps guidance is still with us today, some 20 years later, as evidenced by the average age of the civil works structural documents being over 20 years old. New guidance has evolved on an ad hoc basis for specific use on projects such as the Locks and Dam No. 26 (Replacement) project which, for over 20 years, I had a major role in formulating. Project funds of over $10 million were invested to develop project-specific technology and criteria necessary to validate the design. This cutting edge of technology

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development is currently providing a basis for Corps-wide use through implementation into the Civil Works *Guidance Update Program.*

Evolution In Engineering Technology

The Old Way or the Moment-of-Inertia Method. My mentors, the late patriarchal engineers ("Old Fudds"), among them Bill Stehle (Big Daddy Warbucks), were wizards in using the moment-of-inertia method to analyze everything from internal stresses in concrete and steel, to design of group pile foundations, to the stability analysis of gravity locks and dams.

Equation for Moment-of-Inertia Method for Stresses:

$$f = \frac{P}{A} \pm \frac{MC}{I} \qquad (1)$$

In their younger days these older engineers designed the entire Upper Mississippi River Navigation System (during the 1930's some 28 locks and dams) using this simple little equation, although the numbers crunched were monumental. I also used this method to analyze and design the soil founded U-frame Kaskaskia Lock during the 1960's.

Our Unworldly World of Engineering. During our younger days, structures were assumed as rigid hunks or bodies that did not interact with surrounding media such as soil or water other than as supporting assumed or fictitious soil, fluid, or gravity loads acting on these rigid bodies. These unworldly structures magically appeared at the end of construction for the loads to be instantly turned on. I think we were suffering from a severe case of reality denial in this simplified linear elastic world, but we believed at that time that we were truly applying science to solve technological problems and were real hotshot engineers (we were known as the "young studs"). In reality we were dealing with a world that creeped and flowed, that behaved nonelastically and nonlinearly, whose material properties changed with time and temperature, and which was constructed in little increments over periods of time. Since most structures for design purposes were assumed simply supported, rigid, or elastic, we did not have to worry about material properties or deformations nor follow real-world rules. Analysis calculations were both long and tedious to perform with slide rules, and if you were lucky, you might have had access to a rotary calculator with a back transfer to perform chain multiplication. To analyze a simple frame with sidesway-by-moment distribution took a few days, and to analyze and design a three-dimensional (3-D) gravity dam abutment overflow monolith for sliding and overturning took months and filled several notebooks with calculations. Does anyone remember the "relaxation method" for solving many degree-of-freedom redundant structural systems? What saved us from calamity was that our predecessors had used the same methods for decades and, in the main, produced safe but maybe conservative designs. When there were failures, the authorities made rectifications to either the codes, guidance, criteria, standards, or procedures to preclude future predicaments. Thus, in those days, we were the beneficiary of these previous decades of accumulated experience, mistakes, and modification, and increasingly larger safety factors to deal with our uncertainty. We learned these valuable lessons from our mentors, the older engineers, during these days. I feel an obligation to pass our accumulated knowledge on to the next generation as they did for us, and the Guidance Update Program is a golden opportunity to do so.

The Revolutionary World of Technology. Most of my contemporaries at that time had graduated from college without hearing the word computer mentioned or technology associated with automated analysis and design procedures. I remember that my Professor at St. Louis University (and later to become my boss, Chief of the Structural Section, SLD, Dr. James Cronin) who taught our undergraduate class on structural analysis derived a set of slope stability equations using matrix methods. *We did not comprehend and lost interest* in these concepts then. Dr. Cronin had cautioned us that these techniques were impractical to solve by hand and would require a computer.

This being the 1950's, these computers had not yet been invented. However, we were fortunate that the University of Missouri at Rolla in the 1960's started a night school Civil Engineering Graduate Program in St. Louis about the same time as the District acquired a RCA 301 computer (6,000 words of memory and six tape drives). This became available to the structural designers. I once solved a 49-degree-of-freedom stiffness matrix for deflections, modeling a guide wall, nine-member steel grillage, in about 26 hours on this RCA 301. From these professors that traveled to St. Louis from Rolla, MO, we learned to solve complex plate, frame, grid, and truss systems with many degrees of freedom using computerized stiffness matrix and finite difference methods. Finite element analysis and soil-structure interaction were taught along with the numerical algorithms and programming skills to use computers to solve these problems. We started using and writing programs to automate our work. Those were exciting pioneering times for a young structural engineer, as we relearned and reinvented our structural engineering trade. In 1970, I wrote a pile foundation analysis program using matrix stiffness methods, which is still being used, although it has since evolved into the CASE program CPGA. The example equations shown in Equations 2 through 5 embody stiffness matrix methods, finite difference solutions, and soil-structure interaction. These were revolutionary ideas, not incorporated into our official guidance, which in turn presented difficulties to our review authorities when we submitted designs based on these concepts. These older, wiser, more experienced engineers were naturally skeptical of these new-fangled, unproven notions, since through their experience, they were more comfortable with the old tried and true moment-of-inertia or graphical methods.

Pile-soil interaction stiffness constant:

$$T = \sqrt[5]{\frac{N_h}{EI}} \qquad (2)$$

Relative stiffness factor - pile-soil interaction:

$$B_{11} = K_1 \frac{EI_x}{T_x^3} \qquad (3)$$

Pile-group stiffness matrix:

$$[S]_{6\times6} = \sum_{i=1}^{n} [c]_i[a]_i[b]_i[a]_i^T[c]_i^T \qquad (4)$$

Pile-group deflections:

$$[\Delta] = [S]^{-1}[Q] \qquad (5)$$

Experiences in the Use of Corps Guidance

Pile foundations. We needed realistic pile group foundation design methodology and guidance because we were charged with the design of a billion dollar (current costs) Lock and Dam (L&D) No. 26 (Replacement) across the Mississippi River, now called Melvin T. Price Locks and Dam. Nowhere in the EM on pile foundations (EM 1110-2-2906, "Design of Pile Structures and Foundations," 1958). could we find useful guidance for the analysis and design of groups of vertical and battered piles supporting rigid and flexible structures. This guidance was necessary to design the 30,000 steel H-piles for the foundations for this structure across the Mississippi River. A major program of site-specific, experimental, theoretical, and load testing had been done on the pile foundations for the Arkansas River Project on L&D No. 4. After construction of these L&D's, the engineers, based on their experiences, drafted a new EM for the analysis and design of pile foundations. However, before it was made final, this excellent design group in the Southwest Division was disbanded and the draft was never finished. As a consequence, we in SLD had to recreate the

guidance for L&D No. 26(R), then propose and educate our higher authority on the relevance of this proposed criteria and procedures that incorporated soil-pile-structure interaction (SSI) (Figures 1 and 2). We received comments to the effect that considering soil-pile interaction was too uncertain, and that the soil around the piles should be ignored ($n_h = 0$ in the analysis). Not only being theoretically incorrect (a foundation of vertical friction piles would be unstable), it would also require significantly more pile to satisfy this simplified, over-conservative pile stress design criteria and cost several million dollars more. After much debate, we did convince our higher authority to follow our ways and the proposed SSI criteria and procedures were accepted. Through literature searches and our previous attendance at specialty conferences, we discovered that the University of Texas group under Dr. Lymon Reese were the experts in the field of pile foundation analysis and design. In 1969 we traveled to the University of Texas for a 3-day presentation conducted for SLD by Dr. Reese and his research associates. They presented what they knew about piles from their research and experience from the mostly proprietary R&D they performed on offshore oil platform pile foundations. We had in mind to propose a joint, elaborate research program to develop analysis procedures and design criteria for the foundations for L&D No. 26(R), but since costs were projected to be over a million dollars, this was not approved. I think this was a mistake, because I believe without having certainty in the design criteria, much over-conservatism was built into the design for L&D No. 26(R). If we could have justified raising the allowable pile stresses from the 10,000 psi allowed then to the 12,000 psi we

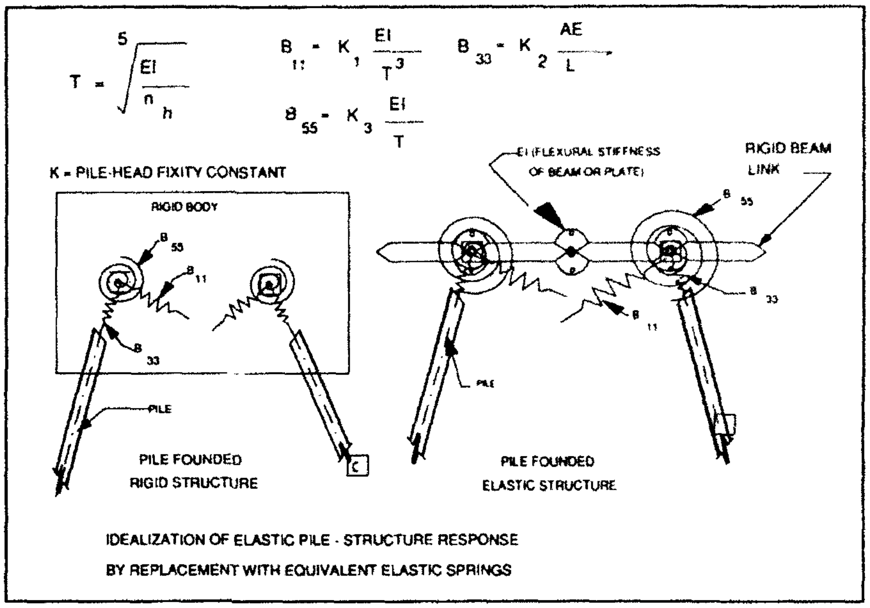


Figure 1. Pile-structure interaction model

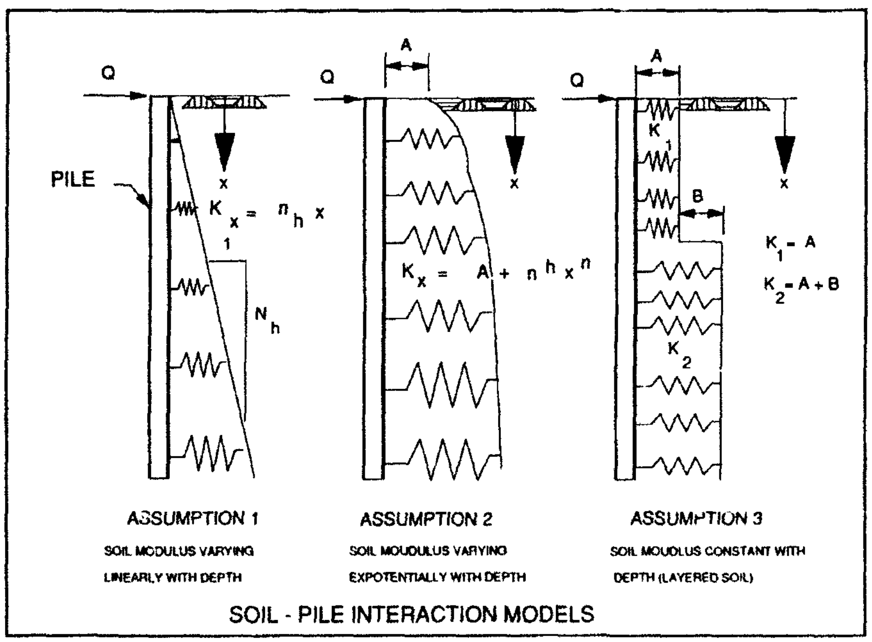


Figure 2. Soil-pile Winkler Spring interaction model

proposed (not approved), this would have paid for the research program many times over. At this time, Dr. N. Radhakrishnan, presently Chief, Information Technology Laboratory, WES, was a graduate student finishing up his Doctoral Thesis on Pile Foundations under Dr. Reese. Dr. Radha tells the story that this particular meeting with the Corps is what convinced him that he should work for the Corps since we had such gigantic, interesting, and challenging problems. Although we lost this tactical issue, I believe we won the global contest by securing Dr. Radha for the Corps. He changed the attitude in the Corps about investing in research which has paid many more dividends in our present engineering knowledge. After Dr. Radha came to WES, he began assisting SLD in developing procedures for pile foundations analysis. At Dr. Radha's encouragement, SLD funded the development of a pile element in the structural analysis program (SAP) finite element method (FEM) by Wayne Jones. This was our initial modification of an FEM coded to include pile element with an elastic plate analysis. To perform the design of the pile foundations for both John H. Overton L&D and L&D No. 26(R), we also created other rigid base and flexible base pile analysis programs. The pile element that was developed for these in-house programs was incorporated into BOEING SAP, GT-STRUDL, and McAUTO-STRUDL. These programs were used successfully to design the 30,000 H-piles used on L&D No. 26(R) and 6,000 H-piles used on the John H. Overton L&D.

Sheet-Pile Cofferdams. The design of sheet-pile cellular cofferdams is another instance where the Corps offered little guidance to the designers because, in general, cofferdams prior to 1970 were contractor designed and constructed. After several major cofferdam failures on the Ohio River Navigation Modernization Project, where the Government ultimately had to assume the costs associated with failures of these contractor-designed-and-constructed cofferdams, in August 1972 the Corps published a new engineering regulation, ER 1110-2-2901, "Construction of Cofferdams." The new policy required that cofferdams of significant magnitude would be Corps designed, thereby the Government would assume the risks associated with these temporary structures placed in treacherous major rivers. Since the Government would assume the risks for these Corps-designed, contractor-constructed cofferdams, the design of these structures would go through the same review and approval process as permanent structures. We, in SLD, had no experience in this area, and we were charged with the task of designing major cofferdams in the Mississippi River for the L&D No. 26(R) Project. At this time, Lehigh University sponsored an industry-wide symposium on the design of pile foundations and sheet-pile cofferdams. I thought this would be invaluable knowledge, necessary to discharge our design responsibility, and I requested attendance at this conference. I was informed by management that we do not sponsor these "boondoggles." After I got over this initial rejection and my hurt feelings, I made a stronger case for our attendance at this meeting and even threatened to take annual leave and attend on my own time. I probably took things too seriously in those days. They relented, we attended, and we picked up invaluable experiences, technical data, criteria, and guidance to be used in the analysis and design of $100 million worth of cofferdams for L&D No. 26(R). At the Lehigh conference, we met all the design and construction experts in this specialized field, including Paul Swatek, and he continues still as a consultant on the L&D No. 26(R) and Olmsted cofferdams. The papers presented at this conference and published in "Design and Installation Pile Foundations and Cellular Structures," along with the experience gained in the Ohio River Division on the Ohio River Navigation Project and the research and prototype instrumentation program SLD sponsored on L&D No. 26(R) have all served as a basis for the development of EM 1110-2-2901, "Design of Sheet Pile Cellular Structures," published 30 Sep 89. Over the years, we improved our knowledge of the nonlinear soil-structure interaction and behavior of these complex structures with a major prototype instrumentation and research program performed for the L&D No. 26 cofferdams (Figure 3). This program will culminate in a doctoral thesis being prepared by Reed Mosher on the three-dimensional nonlinear SSI behavior of cellular sheet-pile cofferdams.

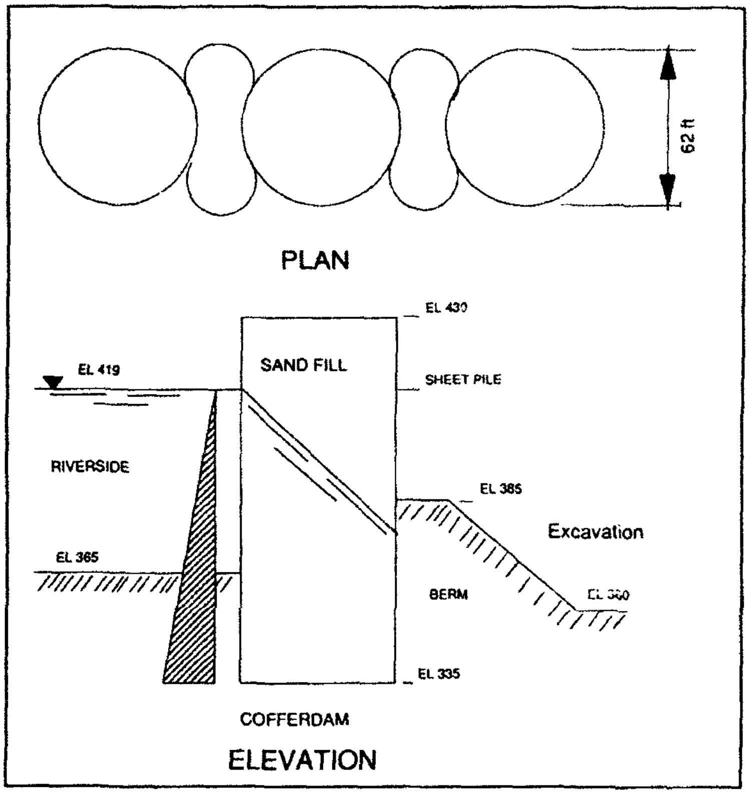


Figure 3. L&D No. 26(R) cofferdam

Reed is now our resident expert on the behavior of sheet-pile walls and cofferdams. I believe, through our efforts using L&D No. 26(R) project funds, we have benefitted the Corps and the engineering profession in the design of cellular cofferdams to the point that today's structural engineer can design these structures with confidence in the ability to predict the behavior and performance for the construction of major future cofferdams, such as those being designed for the Olmsted Project by Louisville District. This project-driven need for realistic state-of-the-practice criteria to analyze and design cofferdams has contributed immeasurably to the official guidance now available for today's designer.

Finite Element Analysis and Design. Finite element analysis and design includes elastic analysis and nonlinear incremental structural analysis.

(1) Elastic analysis. The first design we submitted for review on John H. Overton L&D on the Red River, around 1976, for the U-frame, lock gate bay monolith, pile founded, base slab generated comments from the reviewing authority to the effect that this is an interesting exercise, but they would like to see the hand computations for review of this design. Since we were solving some 30,000 simultaneous equations utilizing 3-D plate finite elements, performing hand calculations would be totally impractical without introducing severe simplifying assumptions to reduce the problem to a simple determinate structure. This was done on a previous project where all the traverse loads were assumed to be carried by the thicker sill section of the monolith. This assumption resulted in six rows of #18 bars at 9-in. centers. The FEM analysis, utilizing the strength contained in the full width of the thick and thin portions of the base slab, resulted in three rows of #18 bars at 12-in. centers, a much more economical and constructable design. Of course, we performed many parametric studies and other verifications such as using the program C-Frame with pile element to validate the final design analysis. This convinced our reviewing authority that our approach was valid, and the design was approved with a comment to the effect that we were to be commended for pioneering new concepts in design procedures. These concepts and procedures developed by SLD were used first on the smaller John H. Overton L&D on the Red River (Figure 4). From this pioneering work led by Roger Hoell, along with Tom Leicht, Joe Hartman, Rich Sovar, and others of that excellent SLD Structural Design group, we were able to successfully transfer this methodology to the design of the much larger L&D No. 26(R).

(2) Nonlinear incremental structural analysis (NISA). However, we were not satisfied that these elastic, gravity-turn-on analysis assumptions accurately described the behavior of these structures in our real world

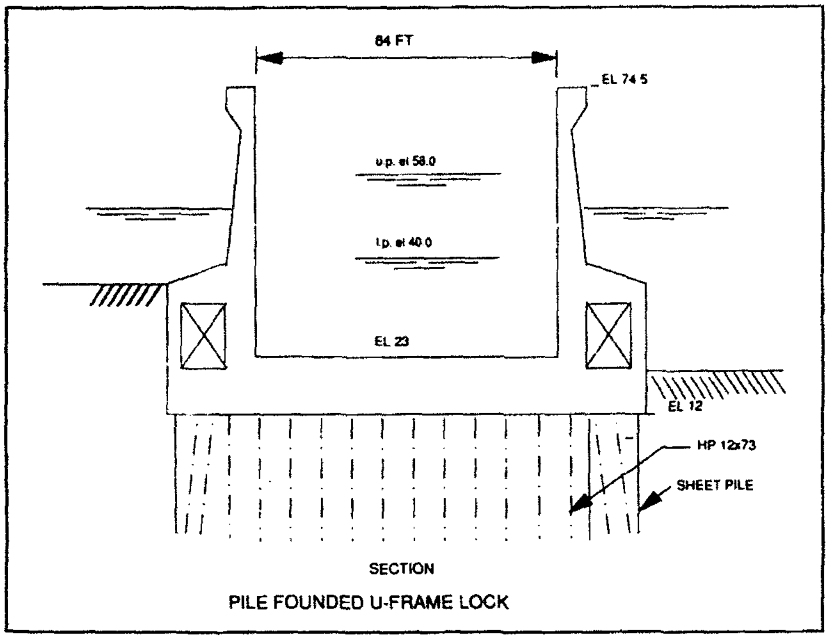


Figure 4. John H. Overton L&D pile founded U-Frame lock

of nonlinear, time-dependent temperature effects, plastic concrete material properties, and incremental construction and loadings. Using L&D No. 26(R) project funds, SLD developed a scope of work and funded development of theory, procedures, and analysis methods which were implemented in a joint effort between SLD (Barry Fehl), WES (Wayne Jones, Tony Bombich, Sharon Garner, Dean Norman), and Washington University (Dr. Kevin Truman). The objective of these studies was to determine if a potential for cracking strains existed in the concrete, to maximize lift heights, and to better predict the interaction with the pile foundation. The results of this project-financed study have resulted in publication of ETL 1110-2-324, "Special Design Provisions for Massive Concrete Structures," published in 1990. Since design funds from L&D No. 26(R) no longer exist, the Corps needs to extend this work with additional funding for model testing, parametric studies, and prototype instrumentation and verification to make these concepts with simplified design guidance available for use by the design engineer in the field offices. Work is ongoing in this area and is described in the paper presented at this conference by Barry Fehl.

The Computer-Aided Structural Engineering (CASE) Experience. One of the most satisfying experiences of my professional career was being a charter member of the CASE project. This project was conceived in 1975 by Don Dressler while working at the Lower Mississippi Valley Division (LMVD). Previous to this time, Don had worked in SLD. I remember during the early 1970's, while I was trying to find a easier, softer way, by automating our structural design with computers, Don would come in at 6 in the morning and derive basic structural theory and procedures from first principles. He was and still is a wizard with pencil and paper. Don discovered $50,000 set aside for computer development in the Division that no one was using. He invited several of us from each of the LMVD Districts to meet in the basement of the Mississippi River Commission Headquarters and figure out how we could use this money to better utilize computers to do our structural engineering work. Out of this basement we formed the LMVD CASD Committee (Computer-Aided Structural Design). Our first project was to automate the design of tainter gates. LMVD funded WES and Dr. Radha to assist in this work, but Dr. Radha did not have a structural engineer available. As a result, we recruited Bill Price. After a painful learning experience, as we were plowing some very new ground trying to develop a black box combined analysis and design package, Bill produced the program TGDA for the analysis and design of tainter gates. This program was used successfully by Milan Hornak to design the tainter gates for L&D No. 26(R). Another project, not so successful, was the PILEOPT (Optimization of Pile Foundations). We had two excellent researchers from the University of Alabama, who spent blood and

tears for a couple of years on this project, but the technology and computer power were just not there to optimize this problem which contained 100's of piles and more than six independent variables. However, we learned what our limits were, and if we resurrect this project I think we can solve this problem with the theory and supercomputers available now.

Richard Armstrong, another SLD graduate, was at that time the Chief of the LMVD Technical Branch. Since he thought what was good for LMVD would be good for the Corps, he, along with Dr. Radha and Don Dressler, organized a Corps-wide structural computer conference in New Orleans during September 1975. Over 200 structural engineers were invited to present the computer programs being used in their design offices. At this CADSE conference, we discovered a world of private structural computer stock in their desk drawers, a lot of efforts duplicated, and a lot that was unique, but it was all shared, champions identified, and communications between like interests developed. From this conference, needs were identified and the CASE project was initiated. Don Dressler moved on to the Chief of Engineers Office and found more R&D money to fund this project. The project was organized by committees in major areas (i.e. 3-D stability, bridges, T-walls, steel structures, buildings, sheet-pile structures, etc.) with engineers from the Districts, Divisions, Laboratories, and OCE serving as committee members. They were tasked with identifying needs, identifying what was available in house or commercially to fill those needs, and identifying gaps where future development was needed. To develop a computer program, a plan is needed, guidance criteria are needed, computer programmers are needed, and user manuals are essential. We quickly realized that guidance criteria were sadly lacking, out of date, and not orientated for computerized technology. I was personally associated with the CASE committee for 3-D stability. After many heated and loud discussions about differing viewpoints on acceptable stability criteria between the 3-D stability committee members, including Bill Kling and Bill Holthem, Fred Tracey would distill the essence and formulate an esoteric program which included all our options. The CASE committees probably spent the first few years in developing the criteria documents before serious computer programming could be accomplished. These criteria documents have served as a starting point for some of the guidance that is presently being published in the Guidance Update Program.

We all know the present benefits the Corps' structural engineers have derived from the CASE project through improved tools to automate the analysis and design process. However, the biggest benefit was the communication and sharing of structural engineering knowledge that has developed between structural engineers in the diversified District, Division, Laboratory, and Chief's offices along with experts from universities, consultants, and computer vendors. Structural engineers sitting in isolation in remote offices had an opportunity to communicate with their peers nationwide. I believe this made the Corps into a truly unified organization that can share professional experience, strengths, and hopes with each other. Engineers who have participated in the CASE project have described this very positive benefit of networking with their peers.

The 1988 Structural Conference. Recognizing all these years that our guidance in the structural design disciplines was obsolete and in great need of updating, Dr. Radha put me up to asking the question at the 1988 St. Louis Structural Conference panel discussion on the closing day, "When is the Corps going to invest in updating our structural design guidance?" Apparently Herb Kennon received the message since things began to happen. Herb thought it was such a good idea that he included all the other engineering disciplines, i.e. hydraulics, geotechnical, electrical, mechanical, and all the others. Herb charged Don Dressler with finding funding and staff to get the program moving. Don Dressler, John McPherson, Dr. Radha, and Gen. Kelly approached the Office of Management and Budget for funding and sold them to the tune of providing $25 million over 5 years to update the over 350 civil works guidance documents. Dr. Radha committed the WES Information

Technology Laboratory to provide staff, administrative, and managerial support; thus, the Civil Works Guidance Update Center was organized at WES. Little did I suspect when I asked that question, that I would be recruited as the Program Manager for the Civil Works Guidance Update Program at WES. For a wiser, mellowed out old "Fudd," I am glad I accepted this assignment. The challenge has heightened my excitement for our profession and renewed my commitment to engineering excellence, stirring the inquisitiveness, impatience for change, and desire of the young "stud" to charge ahead with the best in new technology. I believe the Guidance Update Program offers the vehicle to pass on from this generation of engineers to the next generation our accumulated knowledge and wisdom gained from our failures and successes.

Civil Works Guidance Update Program

Civil Works Guidance Update Center

The Civil Works Guidance Update Center (CWGUC) at WES manages the Corps-wide effort to bring over 350 civil works design criteria documents up to the state of the art through the Civil Works Guidance Update Program (CWGUP). This expedited effort sponsored by HQUSACE, with a budget of $25 million over 5 years, reduces the preparation cycle to at most 2 years instead of the past 6-year average cycle. The program is being implemented by using Corps laboratories, major subordinate commands, field offices, universities, and consultants carefully selected for their expertise to transfer technology from experienced engineers, research, and the private industry to the designers. WES has the responsibility for about one-half of these documents, with all six labs involved. The remaining publications are being done by the Engineering Topographic Laboratory, Cold Regions Research Laboratory, Hydrologic Engineering Center, Hydroelectric Design Center, and various other Corps District and Division field offices. Publication and printing services are provided by the WES Visual Production Center. Management and administrative support for HQ is provided by the CWGUC staff at WES (Tom Mudd, Bill Price, and Wayne Dahl).

Importance of Program

The Water Resource Act of 1988 mandated cost sharing with the local partner. Since local interests must fund up to 50 percent of the improvements for local protection, multipurpose, navigation and harbor projects, the Corps' criteria have come under intense scrutiny, as costs of these projects are directly related to the design criteria. The local partners have criticized the Corps for employing out-of-date, too conservative criteria which have priced some projects out of the local-interest pocket books or into unfavorable benefit/cost ratios. The Government Accounting Office has criticized the Corps in an investigation in the early 1980's for out-of-date, too costly criteria, which were not in line with industry standards in example areas of strength and load factor resistance design. Much of the criteria now used by Corps designers has been of necessity created locally based on informal update enhancement to the largely out-of-date official documents. This is illustrated by Figure 5 which shows the average age of 30 percent of the civil works documents is over 15 years versus only 5 percent of military programs technical manuals over 15 years old. The Corps' criteria have been criticized by local interests, the navigation industry, and the engineering profession as being out-of-date, too costly, and damaging to our credibility in being the leader in water resource development. The impact of the program is realized from the world-wide use of the Corps' written design guidance documents by private and public engineers. As the quality of these documents improves, so will the Corps' reputation for excellence.

The Reason for the Update Program

Due to many years of a grossly underfunded civil works guidance program and lackadaisical oversight management of this limited program, the civil works manuals

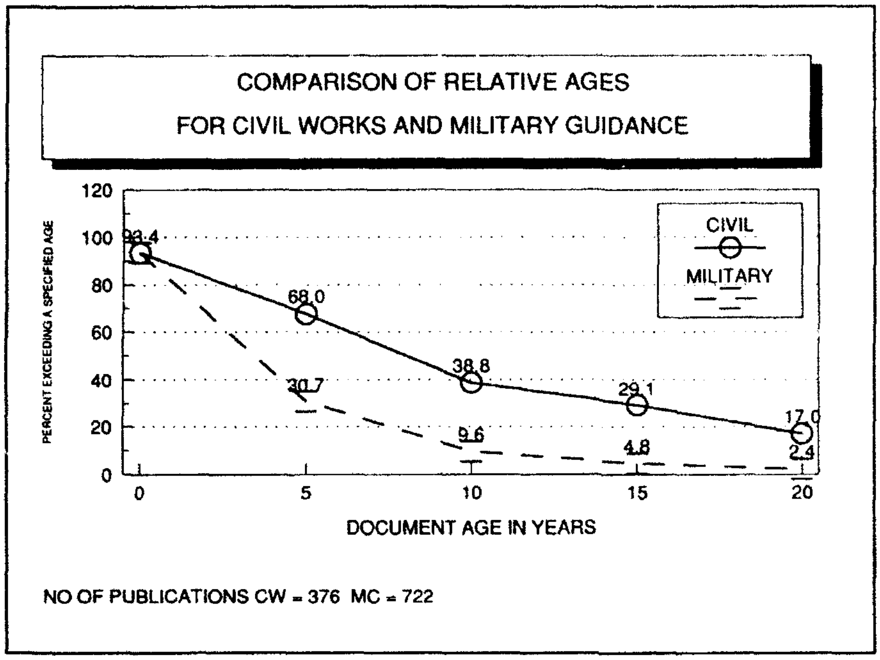


Figure 5. Average age of military program versus civil works

have become outdated. In comparison, the military programs technical manuals have enjoyed adequate funding with the program managed by Huntsville Division in support of the Military Programs Directorate. I have had some personal experience with the amount of time it was taking to prepare an engineering manual on pile foundations. Don Dressler appointed several engineers from the New Orleans and St. Louis Districts to prepare this manual, beginning approximately in 1982. Don had limited funds, only enough to fund a meeting or two a year. We tried to bootleg the work back in our respective offices, but since we could not charge to this work item, it took a low priority, and, consequently, the work progressed painfully slow. This EM was finally published this year under CWGUP. Figure 6 shows the comparison of the civil works versus military program funding over the 5 years previous to the CWGUP. As can be seen in this figure, the lack of funding and management emphasis has seriously eroded the civil works update. This is the justification that was presented to the Office of Management and Budget to obtain the $25 million funding to correct this deficiency. In addition, the Director of Civil Works has placed the highest management priority on this program.

Guidance Program Policy

Driving Force

The driving motivation behind the CWGUP is to reduce the average age to less than 3 years over the 5-year program. At the start of the program, the average age of all civil works documents was 13 years, with the structural documents over 20 years old (Figures 7 and 8).

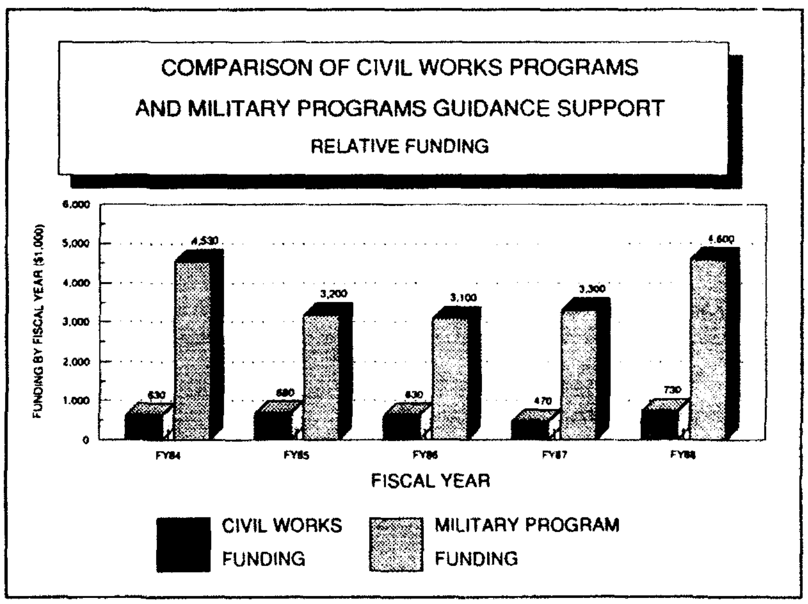


Figure 6. FY84 through FY88 funding of civil works versus military documents

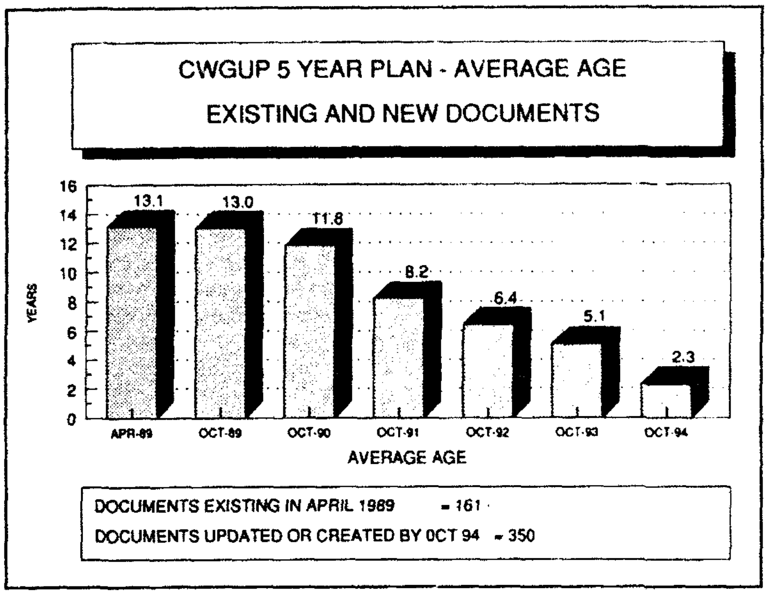


Figure 7. Schedule to reduce average age of civil works documents

Objective

To achieve these goals, business would not be as usual. It was mandated that the products must be developed on time and funds must be expended in the fiscal year that they are received. As an objective, production rates were established for publications as follows.

Engineer Manuals	24-30 months
Engineer Technical Letters	12-18 months
Regulations and Circulars	6-12 months

Coordination

Guidance will be developed from established sources in industry, the engineering profession, established standards, and building codes. The Corps will not duplicate these efforts and will adapt these industry standards in recognition that the Corps has and will have cost-sharing partners, some of which will have their own guidance. Corps guidance will only be developed in those areas where industry standards are not available or are not adaptable to our specialized work for the design of hydraulic structures. The CWGUC is actively coordinating in-house development activities with industry and professional standards sources.

Implementation

The work will be accomplished using one or all of the following methods.

- Field review task groups from lead Districts/Divisions (Similar to CASE Task committees)
- Corps of Engineers laboratories and engineering centers
- Retired experienced Corps structural engineers
- Consultants from universities, other agencies, & industry

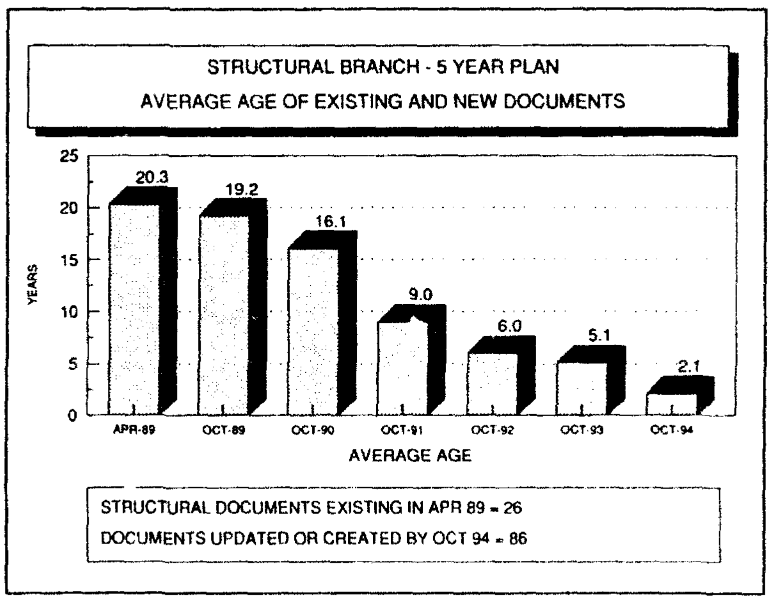


Figure 8. Scheduled reduction in average age of structural documents

- Civil Engineering Research Foundation (CERF) access to ASCE technical committees for peer review and expert *consultation*

Status of CWGUP

Plan

The CWGUP began in April 1989. At that time the HQUSACE Civil Works Directorate was responsible for 161 existing documents that have been determined as candidates for update. During the 5-year plan, 189 new documents will be created to exploit new technology, with 41 interim and existing documents canceled or rescinded because of obsolescence or replacement. Thus by the end of the program in FY94, a total of 350 documents will be created or updated reducing the average age of these documents to 2 years as shown in Figure 9. Figure 10 illustrates the candidate guide specifications, engineer manuals, regulations, technical letters, circulars, pamphlets, and handbooks that have been identified for the update program during the 5-year plan with the type documents scheduled for delivery through 30 September 1991 and the ones delivered to date.

Results

Figure 11 shows the overall results achieved to date. We are meeting about 80 percent of our objectives in the preparation of technical drafts. Performance has been less successful for review and approval and consequent publishing and printing of the documents due to lack of HQUSACE staff and lack of suspense and coordination procedures. These deficiencies are being corrected with hiring of

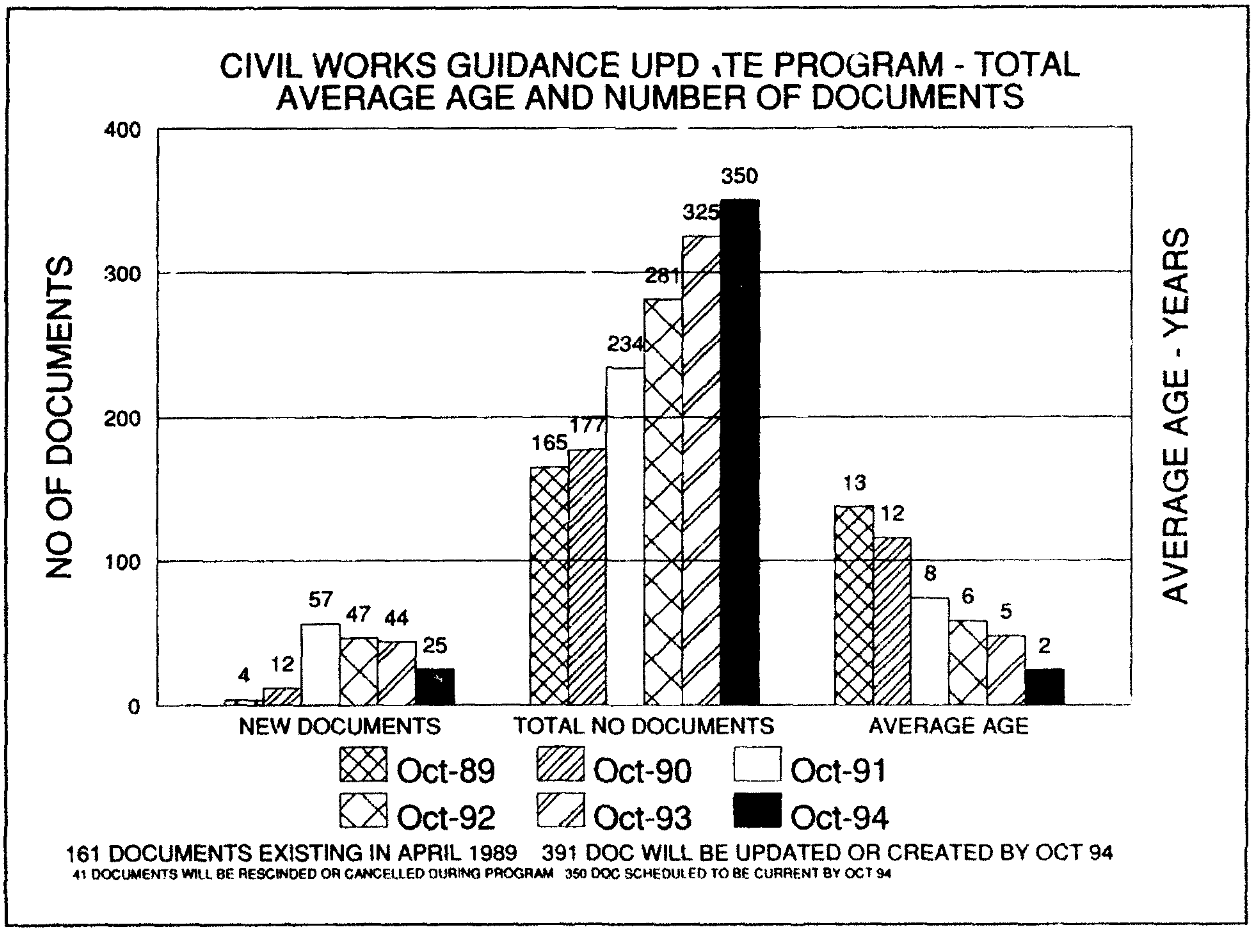


Figure 9. Number and age of existing and new documents in 5-year plan

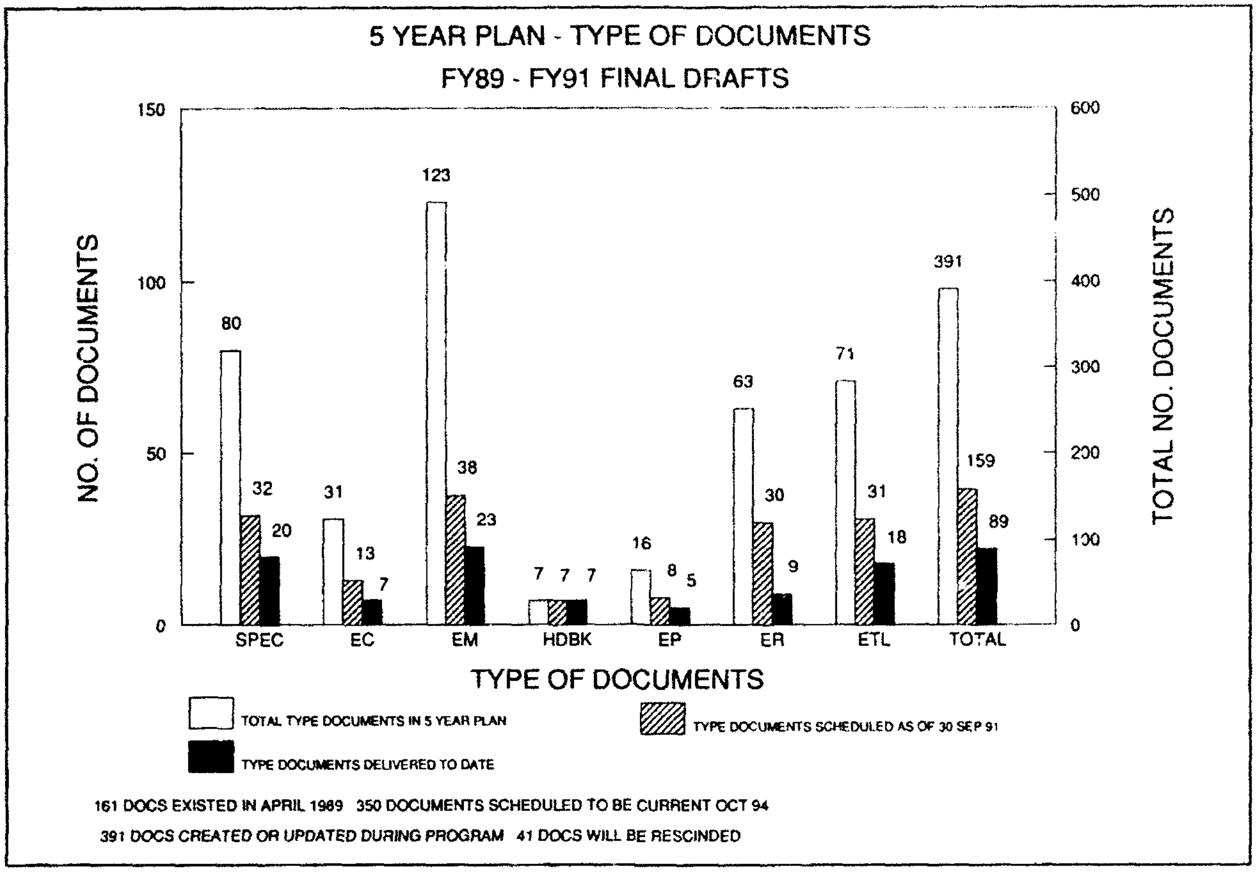


Figure 10. Type of documents in 5-year plan

additional staff and implementation of a tracking, coordination, and suspense system to push the documents through the approval process. Of the 391 documents scheduled for the 5-year plan, 89 final drafts have been forwarded to HQUSACE and 46 documents have been printed and distributed. During FY91, 137 documents are being worked on with 70 final drafts scheduled to be forwarded for technical approval.

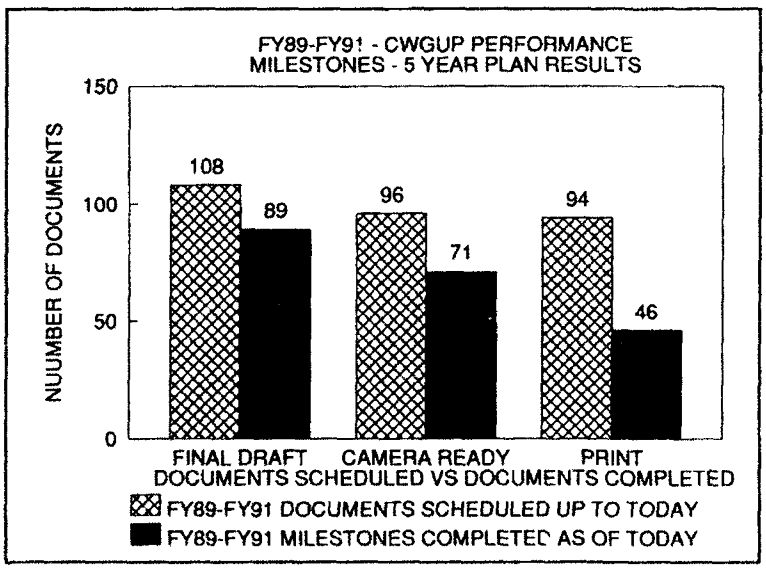


Figure 11. Performance of the CWGUP, FY89 through FY91

CWGUP Organization

Headquarters

HQUSACE has the authority and responsibility to determine policy, criteria, standards, and guidance for the engineering function throughout the Corps. This program falls under the responsibility of John McPherson, Acting Chief of the Engineering Division, Civil Works Directorate

comparing the test results with the arch without Teflon. The first arch tested, which did not have Teflon at the soil-structure interface, was designated as arch S-1. The arch with two Teflon layers at the soil-structure interface was designated arch S-2.

Experimental Investigation

Arch construction details

Construction details and dimensions of the model arch structures are shown in Figure 1. The inside radius of the arches was 1 ft, 9 in., and the thickness of the arch rings was 2 in. Reinforcing steel in the radial direction consisted of D3 wire (area equals 0.0295 $in.^2$) spaced at 2-1/4 in. on center in each face, which resulted in a principal reinforcing steel ratio of approximately 0.008 in each face. Longitudinal reinforcing consisted of D3 wire spaced at 8.5 deg on center and was placed inside the radial steel. A concrete cover of 1/4 in. was maintained over the principal reinforcing steel. After concrete placement and removal of the forms, one of the arches received a 1/32-in. layer of Teflon. The Teflon was glued to the exterior surface and the edges of the arch ring.

Test configuration and procedure

Figure 2 shows the test configuration. The test device is capable of developing pressures up to 3,000 psi. Two layers of Teflon were placed on the inside face of the test chamber to reduce the amount of friction between the sand and the chamber. In each of the two tests, sand was placed to the proper height in the test facility in 6-in. lifts and compacted to provide a uniform support for the model structure. The precast concrete footings were set in place in the chamber, and a steel support for deflection gages was welded to embedded plates in the footings. The arch ring was then lowered into the chamber and placed in the proper position on the footings and grouted in. Transducers for measuring structure loading and behavior were then installed, and sand backfill was placed around and over the arch to a height of 7.5 in. above the arch crown. Steel endplates were used to close the ends of the arches. The ends of the arches and the steel plates were covered with Teflon to provide a Teflon-Teflon interface between the two, thereby reducing the effects of end support as the arches were loaded. After closing up the end of the arch, sand was placed in 6-in. lifts and compacted by making four passes with a 23-lb hand tamp having a 6-3/4- by 11-in. foot.

In each test the data type recorder was started immediately preceding the opening of the waterline valve used to fill the test device with water. The time required to fill the water chamber was approximately 20 minutes. A relief plug at the top of the water chamber indicated when the chamber was full at which time the waterline valve was closed to allow closing of the relief valve. The pump was then started and the pressure in the water chamber was increased very slowly to load the soil surface. As each test proceeded, a plot of water pressure versus arch-crown deflection was monitored to provide a means of determining when a terminate the test.

Instrumentation

Thirty channels of data were recorded on magnetic tape in each of the two tests on a 32-channel Sangamo Model III FM magnetic tape recorder. The data for each channel were later digitized, processed, and plotted. The instrumentation layout for both tests is shown in Figure 3.

Two water-pressure gages (Kulite Model HKM-375) were used to record the pressure applied to the soil surface over the arches. Two gages were used so that if one malfunctioned, data from the backup gage could be used. One of the water-pressure gages was used as a reference channel against which all other data were plotted.

Nine interface pressure (IP) gages (Micro-Gage Model P-302) were mounted around the arch ring at approximately every 22.5 deg to define the pressure distribution around the arch. The gages had a range of 1,000 psi.

publishing, and printing the guidance documents. The latest technology is employed by VPC using Ventura desktop publishing, Intergraph CADD system, and PostScript printing capabilities. All documents are preserved electronically in the VPC for retrieval and reprinting. The VPC has created a new two-column format to make the guidance documents more readable. VPC will furnish all new documents in electronic format for distribution on CD-ROM as part of the Construction Criteria Base (CCB) subscription offered by the National Institute of Building Sciences (NIBS). Tim Ables, Chief of VPC, and Jamie Leach, Chief, Editorial Section, CEWES-IM-MV, are the points of contact. The VPC, under a Memorandum of Understanding (MOU) between HQUSACE and WES, provides the following services for the CWGUP on a cost-reimbursable basis:

- Editing and proofreading
- Drafting and illustrations
- *Camera-ready copy preparation*
- Printing and distribution
- Editorial and format coordination between CWGUP and HQUSACE

Civil Engineering Research Foundation (CERF)

Recognizing that the guidance documents will enter the mainstream for standards of cost-shared water resource development criteria, a contract was awarded to CERF to provide peer review of selected guidance documents by the engineering profession. CERF, which is a nonprofit corporation affiliated with the American Society of Civil Engineers (ASCE), has the capability to provide this service through its access to the technical committees of the engineering societies. Thus, the Corps and CERF have formed a partnership to reach out to the engineering profession to obtain independent and objective peer reviews for Corps guidance with the objective of obtaining industry acceptance of this guidance. CERF's mission for the civil engineering profession is to:

- Assist in update of technical guidance used by the civil engineering profession at large
- Help disseminate and implement CWGUP results
- Identify gaps in current research in the design & construction industry
- Help identify industry needs for future guidance updates
- Help facilitate and coordinate efforts for industry, university, and professional access to Corps updated guidance

In addition, CERF is tasked with developing a list of expert peer reviewers and consultants to assist where needed in reviewing or developing guidance documents. Individuals with the needed expertise can be made available from this matrix of experts in the various engineering disciplines by CERF upon request and execution of a delivery order. These expert individuals and reviewers will offer expert guidance on the state of the art for a particular technology. CERF will in turn draw on the ASCE's long and successful history of developing high-quality manuals and standards to include in the Corps' updated documents. CERF will work with ASCE's technical committees to identify qualified individuals to provide advice and make reviews. CERF will also assist HQUSACE in the coordination and tracking of review documents through the HQ approval system. This will alleviate the bottleneck that presently exists in getting guidance documents reviewed and approved by HQUSACE so they may be printed and into the hands of the designers. The Corps' contract with CERF (awarded August 1990) is for 5 years at a maximum of $400,000 per year with specific delivery orders written against this contract. We believe that this is a very important initiative for a Corps-Industry partnership which will vastly improve the Corps' credibility with the engineering profession as a whole and specifically with our local cost-sharing customers. Harvey Bernstein, President of CERF says, "This is a wonderful program to combine the skills

and expertise of civil engineers from academia, government, and private industry to work together on a common project to develop improved design-construct practices for the industry." Jeff Deemie is the program manager for CERF. His office is located in the CERF-ASCE Washington, DC, office and he is prepared to provide quick response and support to HQUSACE staff in facilitating the CWGUP.

Coordination Complexity

The CWGUP is truly a Corps-wide cooperative effort involving most of the laboratories, design centers, Divisions, Districts, and HQUSACE. These are over 100 engineers involved as principal investigators, field review group members, and technical monitors. Also, engineers from the private sector, consultants, universities, and retired Corps personnel are involved. Part of this complexity can be identified as:

- 2 HQ Directorates
- 10 HQ Branches (7 Civil Works, 3 Military Programs)
- 33 Technical Monitors
- 20 Laboratories and Major Subordinate Commands
- 88 Principal Investigators
- Field Review Groups, Consultants, Universities, CERF, Editors, and Printers
- National Institute of Building Sciences (CCB/CD-ROM)
- TOTAL—Over 155 Significant Associations

Structural Guidance Update Program

Statistics

See attachment 1 for a list of the guidance documents sponsored by the HQ Civil Works Structures Branch in the 5-year plan. As seen in Figure 12, 92 documents are funded for updating or creation during the 5-year plan. Six documents are interim and will be replaced or canceled with 86 structural documents current as of October 1994.

The types of documents are shown in Figure 13. The performance of meeting scheduled milestones for the documents completed to date is shown in Figure 14.

Of the documents assigned to the Structures Branch, 88 percent of the milestones for the technical preparation have been met. However, due to a shortage of staff (unfilled positions) in the Structures Branch, less than 50 percent of the milestones have been met for printing and publishing the documents so that they can be placed in the hands of the designers. This is being rectified at present with recent hires and the assistance of CERF to track the documents through the HQ review and approval system.

Advantage of Structural Guidance Updates

You may ask how the updated structural guidance is going to benefit the structural design engineer located in District structural sections and how it is going to help the reviewers in the Division structural offices. Before guidance is made final, input and advice will be obtained from the designers and reviewers. Most structural guidance that is being developed to introduce new technology will be published in engineer technical letters and engineer circulars to allow field use and comment prior to formalizing into an engineer manual or regulation. A preview of coming attractions may be helpful to describe the benefits to the structural designer for this program.

- Final guidance for strength design of hydraulic concrete structures in conformance with American Concrete Institute Building Code
- Load factor resistance design for hydraulic steel structures in conformance with

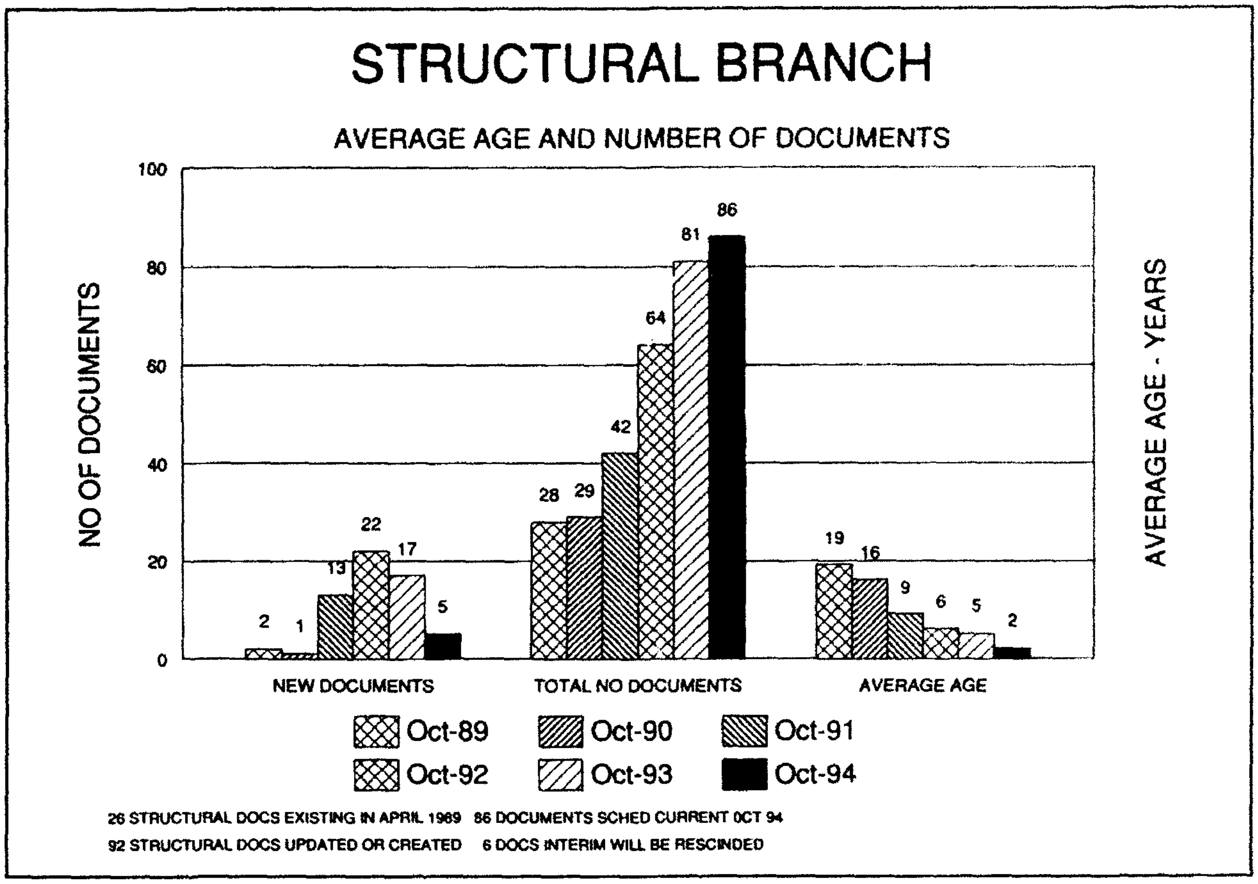


Figure 12. Number of new and updated structural documents in 5-year plan

the American Institute of Steel Construction Code

- Seismic analysis and design, guidance and policy for intake towers, roller-compacted concrete dams, pile foundations, and U-frame locks
- Policy and guidance for life cycle design and evaluation of materials
- Inspection and evaluation of bridges, underwater concrete structures, and steel gates
- Nonlinear incremental structural analysis of massive concrete structures
- Probabilistic evaluation methods and guidance for existing navigation structures
- Probabilistic design methods and criteria for structures and pile foundations
- Fracture analysis of concrete and steel structures
- Authority for structural design responsibilities redefined to the technical engineer from the project management system
- Analysis and design of arch dams, navigation locks, flood control channels, local protection structures, gravity dams, pile foundations, intake towers, sheet-pile structures, and roller compacted concrete structures

Value of the Structural Update Program

Thus, the benefits gained from firm guidance supported by HQUSACE will lead to a better understanding of what is desired from

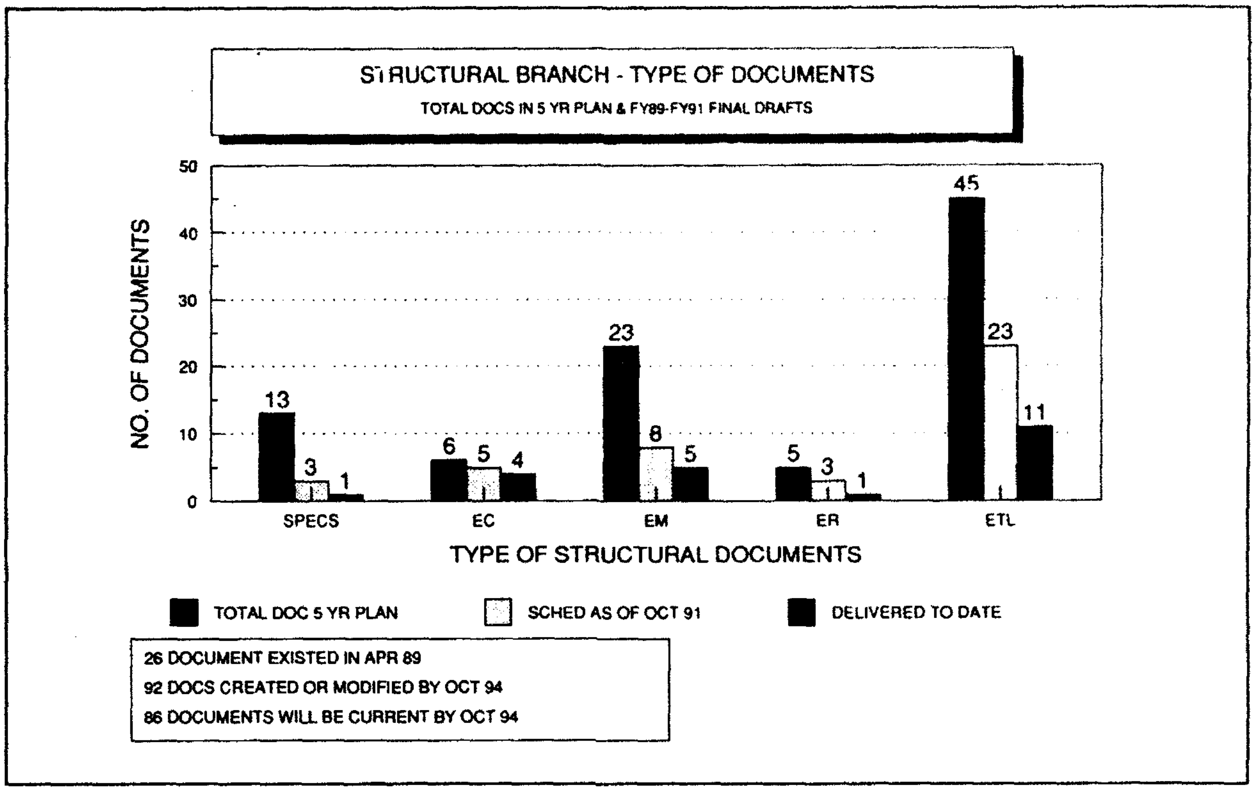


Figure 13. Types of structural documents in 5-year plan

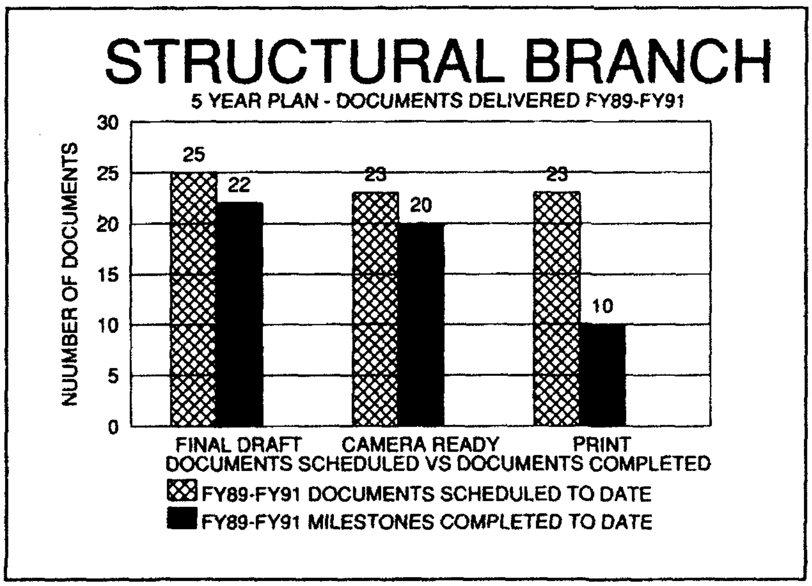


Figure 14. Structures Branch performance

the field offices in terms of quality, work effort, design documentation, and level of engineering so that the designers can properly plan, scope, schedule, and budget for the required work efforts. Advantages of uniform, up-to-date guidance are:

- Less duplication of effort - multiple investments for like development of project-specific design criteria not necessary
- Continuity of institutional engineering knowledge - passed on to next generation of engineers
- Published guidance can function as how-to instructions for analysis and design of Corps structures
- Uniform design and criteria standards between Districts & Divisions for projects that transcend local boundaries (i.e. inland waterway navigation system)

- Project-specific design guidance developed with project funds not lost and available for use by Corps
- Consistent criteria for development of project designs, costs, and benefits for priority of national investment of water resource funds
- Uniformity in construction specifications preventing contractor bid confusion and potential for increased claims
- Availability of valuable project-generated R&D data and criteria to the Corps and the engineering profession as a whole
- Centralizing development of criteria in HQUSACE is cost effective as sufficient resources can be mobilized to generate guidance once rather than every time a new project is formulated
- Designer has available guidance of what is expected from the reviewing authority with instructions on procedures for the design process
- Guidance is available to the designer before project formulation rather than generated after project schedules and costs have been committed

Current Issues

Funding

The CWGUP is funded at $4.5 million per year, with $2 million from "Bill Back" Revolving Fund account and $2.5 million from the General Expense Civil account. This money has been fenced off, although there have been attempts to reduce some of the funding so we are continually having to justify the importance of this program to the Corps. These funds have been provided, as promised, for the previous fiscal years (1989-1991), although somewhat late and in quarterly allotments. A restriction was imposed on the CWGUP in that funds would have to be expended in the same fiscal year as they are received. This has made it very difficult to schedule the work in the labs and major subordinate commands, since most of the documents are planned for a year-round effort. With the funding distribution from HQUSACE being late, and then the funds disappearing on September 30, has caused disruption and slippage has occurred in the program. The Director of Civil Works has personally addressed this problem, and hopefully funds will be allocated from HQUSACE on schedule in October FY92. If the designers feel that the funding for the CWGUP is of benefit for quality designs, then let your management know, so we can maintain a high priority on this effort.

SPECSINTACT

This is an automated specification processing system developed by a joint effort between NASA, NAVFAC, and the CORPS. It is available on the NIBS CCB (National Institute of Building Sciences Construction Criteria Base) CD-ROM subscription. Through the SPECSINTACT system most government and industry guide specifications can be referenced and updated. The Corps' Military Programs proponent for guide specifications, Rodger Seeman, has placed all the Military Programs guide specifications into this system under the administration of the Huntsville Division. The civil works guide specifications were not included in this program because in the reorganization of the HQ Engineering & Construction Directorate into separate Civil Works and Military Programs Directorates in 1988, the proponent position for guide specifications remained in the Military Programs, and no comparable position was created in Civil Works. Consequently, the existing and recently created guide specifications in the CWGUP have not been entered into this system. Presently, CECW-ED is drafting an engineer regulation to place this responsibility under the Structures Branch. It is planned to assign George Gibson as the technical proponent for civil works specifications. His important task will be to find the funding to convert the civil works guide specifications, first into the three-part CSI (Construction Specifications Institute) format, if not already done, and then into the automated SPECSINTACT format. I believe that the specifications are

among the most important tasks and responsibilities that should be defined and assigned during the project formulation stage (Feasibility Report). This is as important as the design formulation and the cost estimate. SPECSINTACT can allow this important identification of specifications outlines for planning and assignment throughout the design process.

Publishing and Printing

Another issue that has created confusion and difficulty is the funding for the preparation of camera-ready copy and subsequent printing. The preparation of the camera-ready copy was to be financed out of the funding assigned to each document; however, when the CWGUP was initiated, the priorities were placed on producing the technical work into a final approved draft. Carry-over funds into the next fiscal year were not allocated to prepare the camera-ready copy. Now that a significant number of documents are at this stage in the pipeline, it has become more difficult to find overhead funds to perform this task. Traditionally, the printing of the documents by HQUSACE was funded out of a separate overhead printing account. When the printing function was transferred to WES by the MOU in December 1989, it was assumed that this funding arrangement would continue. The CWGUC requested $100,000 per year of these printing funds to perform the printing of the guidance documents. The CWGUP received $20,000 in FY90 and nothing in FY91. When pressed on this issue, HQ decided that the printing would have to be funded out of the $4.5 million provided to CWGUP. This will mean the money available for technical preparation will be reduced. To accommodate this change in policy, 8 percent of the CWGUP funds will be reserved for preparation of the camera-ready copy and printing, with this money placed in an overhead account. This will mean that the technical activities will be stretched out, slipping the program into FY95. In addition, planning is being initiated to fund maintenance of the guidance documents after the CWGUP has been completed.

National Institute of Building Sciences (NIBS) Construction Criteria Base (CCB) and CD-ROM

One of the objectives of the CWGUP is to make all the guidance available to the design engineer on the engineer's desktop microcomputer. This can be accomplished by placing the guidance documents on the NIBS CCB CD-ROM. A single CD-ROM disk can hold up to 128,000 pages of information. The NIBS CCB presently consists of four CD-ROM disks that contain most government and industry standards (such as American Society for Testing and Materials standards). Also included are capabilities for CWGUP guidance documents with graphics, CADD design files, MCASES cost estimating system, SPECSINTACT, and executable CASE programs. All this information is available through an $600-$800/year subscription to the CCB. All the engineer needs to access this world of information is a microcomputer with a CD-ROM reader. However, this mass amount of information can overwhelm a person without an intelligent method to search for relevant information. Presently the system can perform word or phrase searches, but this is not very efficient. Ideally, we need an intelligent hypermedia technology, such as HYPERTEXT or some other type of expert system. The CWGUC in partnership with NIBS has submitted a CPAR proposal to accomplish this task. The joint funding requested would be approximately $237,000. It is estimated that the annual savings for the hypermedia system would be on the order of $3 million per year through increased productivity in immediate access by the engineer to current design guidance and standards. The proposal is presently at HQUSACE for evaluation and approval.

Conclusions

Assessment of Program

The CWGUP has had some start-up problems, and slippages have occurred, but these

problems are being rectified. Presently the CWGUP is performing at about 80 percent for preparation of the technical drafts and at about 50 percent for printing documents so they can be placed in the hands of the designer users. The poor performance for pushing the documents through the HQUSACE review and approval system is being rectified with additional hiring or recruiting of staff and with CERF providing assistance in coordinating the approval process with a suspense tracking system. Some 25 to 30 individuals have to sign off on an EM before it can be signed by Col. Herndon, Chief of Staff. Another difficulty has been that since the start of the CWGUP, much of our institutional knowledge and experience has walked out the door due to retirement of key engineers. Through the arrangement with CERF, we hope to bring these elder statesmen back long enough to record their knowledge, and experiences to be included in the Corps guidance, and subsequently to be passed on to those that follow.

Future Technology Enhancements

Don Dressler and the CWGUC are continually searching for new technology to make the design engineer more productive and to relieve some of the paperwork burdens. A study that is popularly reported in many publications estimates that a design engineer devotes only about 35 percent of available time for technical work, the rest is preempted by meetings, paperwork, and all those other demands made on our time (Figure 15). Off in the future, we are thinking toward providing enhanced tools on high-power desktop engineer work stations to improve this productivity ratio. New technology includes incorporating on the CD-ROM, automatic calculations of design procedures directly into the design documents, CADD design files for standard drawings and details, material and specification definitions, and quantity and cost estimating links. With integrated engineering-data base-CADD-guidance-graphics visualization-hypermedia-expert

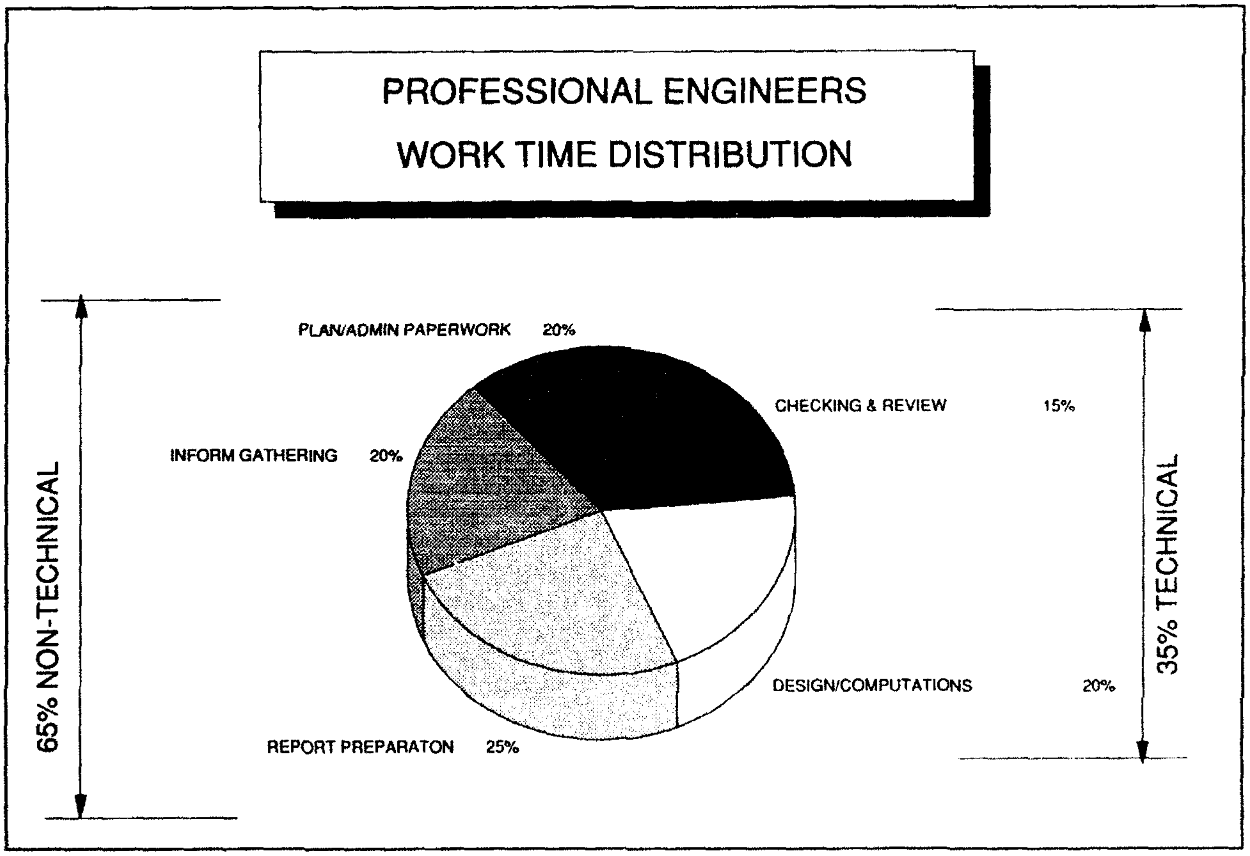


Figure 15. Professional engineer technical efficiency

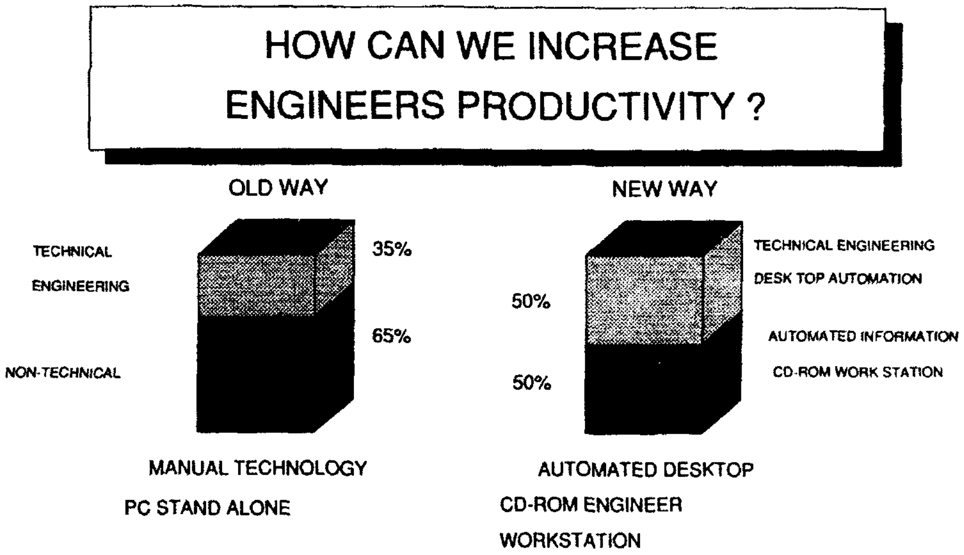
systems and fiber optic/networking for moving large amounts of information to and from remote main frame/supercomputer sites, we hope to provide the capability to improve the technical productivity ratio to a 50-50 split as projected in Figure 16.

Challenge for the Champions

Where this Engineer is Today

I believe that "*Quality Design Demands Quality Engineers*" (*ASCE Magazine* 1985). We need designers who can couple experience and judgment with computer-aided analysis and design and other high-technology procedures to produce designs that function as planned, are aesthetically pleasing, conserve resources, have minimum life cycle costs, and meet project budgets and schedules. In the old days of slide rules, the designer worked in a two-dimensional, linear elastic world with rules of thumb and engineering judgment based on precedent. The engineering manager probably acquired the Chief's position after many years in technical design before assuming full professional responsibilities as an independent senior engineer decisionmaker.

Today's designer works in a very different world. This world is real in space and time, is nonlinear and nonelastic, is static and dynamic, flows, creeps, and has material properties that change with age and deformation. With computer simulations, it is possible to describe and analyze these real world events and further obtain optimum design by parametric investigations. Thus, today the designer can quickly gain the experience in natural behavior that a generation ago he could only have acquired in a lengthy trial-and-error process, with sometimes the errors showing up after construction.



AUTOMATED ENGINEERING/INFORMATION MANAGEMENT SYSTEMS

- DESK TOP ENGINEER WORK STATIONS
- ENGINEERING & CONSTRUCTION INTEGRATED DATA BASE/CADD
- CD-ROM DATA BASE HYPERTEXT/HYPERMEDIA/EXPERT SYSTEMS
- FIBER OPTIC INFORMATION TRANSFER/NETWORKING REMOTE SITES
- ON LINE MAIN FRAMES/SUPER COMPUTERS/SCIENTIFIC VISULIZATION

Figure 16. Tools to improve engineers' productivity

To effectively model, analyze, and interpret a design using the latest technology, the designer must think, act, and work smarter and more independently than ever before. The technical design decisions are being made at lower levels of the organization, while the project managers are more concerned with the business, the schedules, and the budgets. Unfortunately this is where recognition, advancement, and compensation are found, and we lose many of our best designers to this end of the organization.

Thus, the designers that choose to remain need all the help they can get to perform the essential function of the Corps of Engineers and to be the best they can be at engineering. Today's designer needs the best tools, the best technology, and the best guidance to do the best job. That is why I believe the most effective civil works guidance program that we can devise is essential for the survival of our designer to perform quality engineering. This is where I want to be, at least today.

Yesterday is History
Tomorrow is but a Dream
Only today is Real

or stated in other terms:

It is very difficult
to survive in a world of meat eaters
when you are wearing meat

Breakfast of Champions

Since, at this early hour you have foregone breakfast to listen to my reminiscences, I challenge you all, as professional structural engineers, with the opportunity to participate in the guidance update program. Each of you is a champion, as evidenced by your active participation in this structural conference which shows a willingness to learn, share, and contribute your knowledge to the betterment of structural engineering. As champions, we need your advice and contribution to this program. I have always believed that the best engineers to write guidance are designers, not researchers that create technology. The engineers have experience in translating new technology into the designs that can be constructed and have been around long enough to experience the joys and pains through the execution of designs and have learned to do things differently and better on the next job. We need your experiences, strengths, and hopes in written guidance to pass on to the next generation. We can't keep it unless we pass it on. If you want to volunteer to write guidance, serve on field review groups, or even offer suggestions where new guidance is needed, call Donald Dressler, CECW-ED, 202-272-0220, or Thomas Mudd, CEWES-IM-D, 601-634-4383, and we will get you involved.

ATTACHMENT 1 CIVIL WORKS GUIDANCE UPDATE CENTER – CIVIL WORKS GUIDANCE UPDATE PROGRAM 27–Jun–91 PREPARED BY: THOMAS J. MUDD, P.E. CEWES–IM–DG

5 YEAR PLAN – FY89 THRU FY94
STRUCTURAL BRANCH

FY89 PRODUCT
* FY90 PRODUCT
@ FY91 PRODUCT

NO.	TYPE	PUB–NUMBER	TITLE	OCE PROPONENT	OCE NAME	POC LEAD FOA	POC NAME	5–YEAR TOTAL	PRINT & DISTR	MS	COMMENTS
1	EM	1110–2–2502	Retaining and Flood Walls........................ #	CW–ED–LG	GUTHRIE	CEWES–IM–DS	PACE	27	29–Sep–90	8	*** Printed & Distributed
2	CW	02315	Piling – Steel – H Piles......................... #	CW–ED–GG	GIBSON	CEWES–IM–DS	JAEGER	9	12–Feb–90	8	*** Printed & Distributed
3	ETL	1110–2–324	Special Design Provisions for Massive Concrete #	CW–ED–DD	DRESSLER	CEWES–IM–DS	JAEGER	20	12–Feb–90	8	*** Printed & Distributed
4	ETL	1110–2–322	Retaining Wall Design and Anaylsis.................... #	CW–ED–LG	GUTHRIE	CEWES–IM–DS	PACE	1	23–Jan–91	8	***PRINTED & DISTRIBUTED
5	ETL	1110–2–XXXX	LRFD of Steel Miter Gates............................ #	CW–ED–DC	CHEN	CEWES–IM–DS	JAEGER	27	30–Apr–91	7	CAMERA RDY TO HQS
6	EM	1110–2–2906	Design of Pile Foundations... #	CW–ED–DD	DRESSLER	CEWES–IM–DS	JAEGER	82	15–Jan–91	8	PRINTED
7	EC	1110–2–XXXX	Strength Design of Reinforced Concrete Hydr St#	CW–ED–TL	LIU	CELMV–ED–TS	COX	13	29–Mar–90	8	Draft publ & dist as an EC
7	EM	1110–2–XXXX	Strength Design of Hydraulic Structures ...... @	CW–ED–DD	DRESSLER	CEWES–IM–DG	PRICE	140	28–Oct–91	5	CONVERT EC TO EM
8	EM	1110–2–XXXX	Structural Analysis of Intake Towers	CW–ED–LG	GUTHRIE	CENPP–ED–D	ILLIAS	40	28–Oct–92	0	FY89 PROD RESTART W/ETL
9	ETL	1110–2–XXXX	Struct Des of Freeboard Walls for Earth Emb.....*	CW–ED–LG	GUTHRIE	CEWES–IM–D	PRICE WP1	27	30–Apr–91	7	CAMERA RDY TO HQS
10	ETL	1110–8–17(FR)	Pile Layout to Minimize Interference ....*	CW–ED–TM	MUDD	CEWES–IM–DS	MOSHER RM3	52	30–Apr–91	7	CAMERA RDY TO HQS
11	ETL	1110–2–XXXX	Struct Des of U Channels, Basins & Drop Str......*	CW–ED–LG	GUTHRIE	CEWES–IM–D	PRICE WP2	27	30–Apr–91	7	CAMERA RDY TO HQS
12	ETL	1110–8–16(FR)	Fracture Analysis of Concrete Hydraulic Strs.......*	CW–ED–DD	DRESSLER	CEWES–IM–DS	JAEGER JJ2 JJ(	30	30–Apr–91	7	CAMERA RDY TO HQS
13	ETL	1110–8–13(FR)	Structural Engineering Responsibilities for CW P*	CW–ED–DD	DRESSLER	CEWES–IM–DS	JAEGER JJ3 JJ(	65	30–Apr–91	7	CAMERA RDY TO HQS
14	EC	1110–2–270	Structural Design of Flood Control Channels *	CW–ED–GG	GIBSON	CEWES–IM–D	PRICE WP3	56	17–Jun–91	7	CHG EM TO EC BACK TO HQS
15	EM–	1110–2–XXXX	Concrete Floating Breakwater Structural Design.	CW–ED–TL	DRESSLER	CEWES–SS–A	HALL	75			MORE R&D REQUIRED
16	EM	1110–2–XXXX	Design of Sheet Pile Walls (incl Suppl) – CEWE*	CW–ED–DD	DRESSLER	CEWES–IM–DI	MOSHER RM1 F	269	03–Feb–91	5	SUP–ISSUE AS EC JAN91
17	EM	1110–2–XXXX	Design of Arch Dams (and Supp).................... @	CW–ED–DD	DRESSLER	CEWES–IM–DS	JONES/NAVIDI	190	03–Feb–92		
18	ETL	1110–8–3(FR)	Underwater Inspection of Concrete Structures...*	CW–ED–TL	LIU	CEWES–SC–R	McDONALD	25	15–Feb–91	8	PRINTED
19	EC	1110–2–2103	Details of Reinforcement........................ *	CW–ED–DC	CHEN	CEWES–IM–D	PRICE	0	30–Jan–90	8	PRINTED–INCL W/EC #7
20	ETL	1110–8–8(FR)	Seismic Anal & Des of Intake Towers– PART II...*	CW–ED–LG	GUTHRIE	CEWES–IM–DS	JAEGER JJ6	32	03–Feb–91	6	PREPARING CAMERA READY
21	ETL	1110–2–XXXX	EQ Analysis of Gravity Dams.......................	CW–ED–LG	GUTHRIE	CEWES–SS–A	HALL	5			CANCEL BY HQ NOT APPROVED
23	CW	5560	Steel Flood Closure Gates .... @	CW–ED–GG	GIBSON	CEWES–IM–DG	PRICE	30	03–Feb–92	5	
24	EM	1110–2–2200	Gravity Dam Design.............................. *	CW–ED–TL	LIU	CENPP–EN–DS	TM1,ILLIAS	0	03–Feb–91	7	CAM RDY TO HQS, NO BILLING
25	ER	1110–2–99	Bridge Safety Inspection & Evaluation................*	CW–ED–DC	CHEN	CEWES–IM–D	PRICE WP6	44	30–Apr–91	7	CAMERA RDY TO HQS
26	ETL	1110–8–2(FR)	Anchor Embedment in Hardened Concrete ....*	CW–ED–TL	LIU	CEWES–SC–R	McDONALD	15	01–Oct–90	8	PRINTED
27	ETL	1110–2–XXXX	Seismic Design Provisions for RCC Dams..........@	CW–ED–TL	DRESSLER	CENPD–ED–TE	STROM	103	03–Feb–92	4	
28	ETL	1110–2–XXXX	Life Cycle Perf. of LP Structures.........................@	CW–ED–DD	DRESSLER	CESWD–EN–TE	HARTMAN	78	28–Oct–92	3	$30K CERF DO#3
29	ETL	1110–2–XXXX	Structural Evaluation of Existing Lock Gates.......@	CW–ED–TL	DRESSLER	CESAJ–ED–DS	JAEGER	55	28–Oct–91	2	
30	ETL–	1110–2–XXXX	Fracture of Concrete Coastal Structures	CW–ED–DD	DRESSLER			0			CANCELLED–NEEDS BASIC R&D
31	EM	1110–1–2101	Allowable Stresses (Working and LFRD)...........	CW–ED–DC	DRESSLER	CEWES–IM–DS	CHASTEN	145	28–Oct–92	5	
32	ETL	1110–2–XXXX	Gated Closure Structures for LFPP............ ... @	CW–ED–LG	GIBSON	CEWES–IM–DG	PRICE	33	03–Feb–92	5	
33	ETL	1110–2–XXXX	Finite Element Modeling.......................... @	CW–ED–TL	DRESSLER	CEWES–IM–DS	JONES	35	28–Oct–91	3	
34	ETL	1110–2–XXXX	Design of Mod/Arch Comm Wall Systems..........	CW–ED–LG	GUTHRIE	0	0	0			NOT FUNDED IN FY91
35	ETL	1110–2–XXXX	Design of Permanent Tieback Walls...................	CW–ED–DD	DRESSLER	0	0	0			NOT FUNDED IN FY91
36	ETL	1110–2–XXXX	Str Design Using the RCC Construction Process.	CW–ED–DD	DRESSLER	CENPD–ED–TE	STROM	85	06–Jun–92	3	
37	ETL	1110–2–XXXX	Seismic Design of 3D Pile Groups.................. @	CW–ED–TL	LEICHT	CEWES–IM–DI	MOSHER	140	18–Sep–91	3	
38	ETL	1110–2–XXXX	Design Reliability of Pile Substructurs.................	CW–ED–TL	LEICHT	CEWES–IM–DI	MOSHER	155	06–Jun–92	2	
39	ETL	1110–2–XXXX	Navigation Locks – Feasibility Studies .... @	CW–ED–LG	GUTHRIE	CECW–ED	LEICHT	182	03–Feb–92	1	
39	EC	1110–2–2602	Plan. & Design Navigation Locks..................	CW–ED–LG	GUTHRIE	CECW–ED	LEICHT	70	06–Apr–93	1	$70K CERF DO#4
40	ER	1110–2–XXXX	SPECSINTACT (Management Plan.....................@	CW–ED–GG	GIBSON	CEWES–IM–DG	MUDD	55	03–Feb–92	5	1ST DRAFT ER COMPLETED
41	CW	XXXX	Precast Anchored Retaining Wall Systems.........	CW–ED		0		0			NOT FUNDED IN FY91
42	CE	1304.04	Piling – Concrete – Precast......................	CW–ED–GG	GIBSON	0	0	0			CANCEL–NOT USED BY CORPS
43	CE	XXXX	Piling–Concrete–Prestressed..........................@	CW–ED–GG	GIBSON	CEWES–IM–DG	PRICE	25	28–Oct–91	2	
44	ETL	1110–2–XXXX	Flotation Stability of Anchors & Pile Suppt Strs..	CW–ED	0	0	0	0			NOT FUNDED IN FY91
45	ETL	1110–2–XXXX	Strength Design of Concrete conduits............	CW–ED	0	0	0	0			NOT FUNDED IN FY91
46	ETL	1110–2–XXXX	Underwater Repair of Stilling Basins..................@	CW–EG–TL	LIU	CEWES–SC–R	McDONALD	40	03–Feb–92	2	
47	ETL	1110–2–XXXX	Str Eval of Exst Welded & Riveted Spillway Gate@	CW–ED		0 CESAJ–ED–DS	JAEGER	60	03–Feb–92	0	
48	ETL	1110–2–XXXX	Corrosion Protection for Reinforcement...........	CW–ED		0		30			NOT FUNDED IN FY91
49	ETL	1110–2–XXXX	Design of Anchored Rigid Structures..............	CW–ED		0		45		0	NOT FUNDED IN FY91
50	ETL	XXXX	Piling–Steel–Round..................................	CW–ED		0		15		0	NOT FUNDED IN FY91
51	ETL	1110–2–XXXX	Design Considerations for High Head Slide Gate	CW–ED		0		40		0	NOT FUNDED IN FY91
52	EM	1110–2–2607	Navigation Dam..................................	CW–ED		0		150		0	NOT FUNDED IN FY91
53	EM	1110–2–XXXX	U–Frame Navigation Lock Design..................	CW–ED		0		170		0	NOT FUNDED IN FY91
54	CW	02311	Piling – Timber – Round..........................	CW–ED		CEWES		20			
55	CW	XXXX	Precast Modular Retaining Wall Systems............	CW–ED		CEWES		25			

ATTACHMENT 1A   CIVIL WORKS GUIDANCE UPDATE CENTER – CIVIL WORKS GUIDANCE UPDATE PROGRAM   27–Jun–91 PREPARED BY: THOMAS J. MUDD, P.E. CEWES–IM–DG

5 YEAR PLAN – FY89 THRU FY94
STRUCTURAL BRANCH

NO.	TYPE	PUB–NUMBER	TITLE	OCE PROPONENT	OCE NAME	POC LEAD FOA	POC NAME	5–YEAR TOTAL	PRINT & DISTR	MS	COMMENTS
56	ETL	1110–2–XXXX	Steel Liners for Hydropower Conversions..........	CW–ED				45			
57	ETL	1110–2–XXX	Miter Gates – Rigid Cellular Type....................	CW–ED		CEWES		45			
58	ETL	1110–2–XXXX	Design of Stoplogs/Emergency Closure.............	CW–ED		CEWES		70			
59	ETL	1110–2–XXXX	Slurry Placed Reinforced Concrete Walls..........	CW–ED		CEWES		80			
60	ETL	1110–2–XXXX	Life Cycle Perf of Strs for Hurricane Protection...	CW–ED		CEWES		80			
61	ETL	1110–2–XXXX	Non–linear Structural Analysis....................	CW–ED		CEWES		110			
62	EM	1110–2–2703	Lock Gates..	CW–ED		CEWES		145			
63	EM	1110–2–XXXX	Mon & Eval of Long Term Perf of Str Post Tens S	CW–ED		CEWES		100			
64	ETL	1110–2–XXXX	Strengthening Hydraulic Structures................	CW–ED		CEWES		170			
65	EM	1110–2–XXXX	Structural Stability Analysis......................	CW–ED		CEWES		240			
66	EM	XXXX	Tainter Valves......................................	CW–ED		CEWES		30			
67	ER	1110–2–XXXX	Engineer of Record.................................	CW–ED		CEWES		30			
68	CE	1506	Vertical Lift Crest Grates.........................	CW–ED		CEWES		60			
69	EM	1110–2–2901	Tunnels & Shafts in Rock.........................	CW–ED		CEWES		105			
70	EM	1110–2–4300	Instrumentation of Concrete Structures...........	CW–ED		CEWES		100			
71	EM	1110–2–2902	Conduits, Culverts.................................	CW–ED		CEWES		130			
72	EM	1110–2–2002	Evaluation & Repair of Concrete.......	CW–ED		CEWES		130			
73	EM	1110–2–2702	Design of Spillway Tainter Gates.................	CW–ED		CEWES		130			
74	EM	1110–2–2701	Vertical Lift Gates................................	CW–ED		CEWES		80			
75	EM	1110–2–3001	Planning & Design of Hydropower..................	CW–ED		CEWES		50			
76	CW	05563	Tainter Gates & Anchorage.........................	CW–ED		CEWES		20			
77	ER	1110–2–2901	Construction of Coffer fams........................	CW–ED		CEWES		20			
78	CW	02411	Steel Sheet Piling.................................	CW–ED		CEWES		20			
79	CW	05550	Welded Power Penstocks...........................	CW–ED		CEWES		20			
80	CE	1507.01	Tractor Gates – Broome Types.....................	CW–ED		CEWES		10			
81	CE	03150	Expansion, Contraction & Constr Joints in Concr	CW–ED		CEWES		10			
82	ETL	1110–2–XXXX	Preliminary Design of Drop Structures............	CW–ED		CEWES		45			
84	EM–	1110–2–XXXX	Structural Analysis of U Frame Locks.............	CW–ED		CEWES		0			NOT FUNDED IN 5 YEAR PLAN
85	EM–	1110–2–3104	Structural & Archit Design of Pumping Stations...	CW–ED		CEWES		0			NOT FUNDED IN 5 YEAR PLAN
87	EM–	1110–2–XXXX	Bridge Design......................................	CW–ED		CEWES		0			NOT FUNDED IN 5 YEAR PLAN
88	ER–	1110–2–XXXX	Lateral Earth Pressures – Seismic.................	CW–ED–LG				0			NOT FUNDED IN 5 YEAR PLAN
101	EM	1110–2–3001	Planning & Design of Hydro–Elect Pwr Plants ...*	CW–ED–DD	DRESSLER	CENPD–EN–HD	LOU M	0	03–Feb–91	6	New–Final DRFT TO HQS
102	EM	1110–2–XXXX	Design Response Spectra for Structurs ....@	CW–ED–GG	GUTHRIE	CEWES–IM–DS	EBELING	90	19–Oct–91	4	
103	EC	1110–2–265	Engineering & Design (FY89 PRODUCT)...........#	CW–ED–DD	DRESSLER	CEWES–IM–MV	ABLES	61	30–Sep–89	8	PUBLISHED IN FY89
104	EC	1110–2–265	Engineering & Design(FY90 PRODUCT).............*	CW–ED–DD	DRESSLER	CEWES–IM–VP	ABLES TA1	15	03–Feb–91	7	$15K Unfunded(Tim & Cissy)
105	ER	1110–2–265	Engineering & Design(FY91 PRODUCT).............@	CW–ED–DD	DRESSLER	CEWES–IM–VP	ABLES	10	28–Oct–91		
106	ETL	1110–2–XXX	Navigation Structure Condition Anal & Eval...... @	CW–ED–DD	DRESSLER	CEWES–IM–DS	LEGGETT	155	28–Oct–91		OVERBUDGET ITEM FOR FY91
107	ETL	1110–2–XXXX	Seismic Analysis & Desing of U–Frame Locks ...@	CW–ED–LG	GUTHRIE	CEWES–IM–DI	MOSHER	70	28–Oct–91	0	
108	ETL	1110–2–1806	Earthqke Des & Anal of CE Prjts (Coord/Revise).@	CW–ED–LG	GUTHRIE	CEWES–IM–DS	EBELING	10	03–Feb–92	0	ADDED TO PROG–FY92 PROD
109	ETL	1110–2–XXXX	Stability of Gravity Walls (Vertical Shear)	CW–ED–LG	GUTHRIE	CEWES–IM–DS	EBELING	10	03–Feb–93	0	ADDED TO PROG–UNBUDGETED
110	ETL	1110–2–XXXX	Design & Constr Techn for Underwater Repairs .	CW–EG–TL	LIU	CEWES–SC–R	McDONALD	10	04–Sep–92	0	
111	ETL	1110–2–XXXX	Planning for Risk Analysis for Water Resource Ir	CW ED–DD	DRESSLER	CEWES–IM–DI	MOSHER	10	03–Feb–92		ADDED BY DRD (USE MP FUNDS)
92 Publications			SUBTOTALS ($1,000)..	CECW–ED				$5,760			

23	SCHEDULED TO PRINT
10	PRINTED TO DATE

NO.	MILESTONES
1	REVIEW SCOPES OF WORK, FUNDING, MILESTONES
2	PREPARE OUTLINE
3	FIRST DRAFT COMPLETE
4	COMPLETE STUDIES, REVISE DRAFT
5	PERFORM TECHNICAL REVIEW BY FRG, EDIT
6	SUBMIT FINAL DRAFT TO HQS FOR APPROVAL
7	SUBMIT CAMERA RDY COPY TO HQS FOR SIGNATURE
8	PUBLISH, PRINT AND DISTRIBUTE

		FINAL DRAFTS TO HQS	
TYPE	NO DOC	FD SCHED	FD DELIV
GS	13	3	1
EC	6	5	4
EM	23	8	5
EP	0		
ER	5	3	1
ETL	45	23	11
TOTAL	92	42	22

Earthquake Implications for the Central and Eastern United States

by
Helen J. Petersen[1]

Abstract

The Loma Prieta earthquake of October 1989 serves as a graphic reminder of the destructive power of ground shaking amplified by soft soils. Communities severely impacted by the earthquake are still in what promises to be a long recovery process. The damage caused by the Loma Prieta earthquake emphasizes the importance of the seismic structural design principles in use today.

This paper looks at the seismicity of the Central and Eastern United States. Major historical seismic events and earthquake source zones are identified, and characteristics of "intraplate" earthquakes are briefly discussed. Factors placing the region at risk including lack of understanding of intraplate seismic activity, uncertainty establishing earthquake frequency and recurrence rates. lack of seismic provisions in building codes, failure to address the seismic retrofit of existing buildings, and vulnerability of bridges and highways are discussed in general terms. The paper concludes that the Central and Eastern United States are at risk of an earthquake and that even a moderate earthquake depending on the magnitude, epicenter location, and focal depth could cause significant damage.

Introduction

On 17 October 1989 at 5:04 P.M. Pacific Daylight Time, an earthquake 7.1 M_s (surface wave magnitude) occurred along the San Andreas Fault. The epicenter of the earthquake was located in the Santa Cruz mountains, approximately 10 miles east-northeast of the city of Santa Cruz and about 60 miles southeast of San Francisco near Mount Loma Prieta. The Pacific Plate moved 6.2 ft to the northwest and 4.3 ft upward over the North American Plate. The focal depth of the earthquake was about 18 km, which is much deeper than the 8- to 10-km range expected for earthquakes in the region (The State/Hazard Mitigation Team 1990).

There were 62 fatalities as a result of the earthquake, and the property damage estimates approach $10 billion, but fortunately several factors combined to minimize the loss of life and property due to Loma Prieta.

- The epicenter was in an area of low population density.
- The period of heavy ground shaking was relatively short, less than 15 sec. This was due primarily to the bilateral rupture mechanism.
- The time of the earthquake coincided with the World Series. Many commuters

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had adjusted their schedules; consequently, the number of people at great risk was reduced.

- California had adopted seismic building codes and practiced earthquake resistant design. While not all buildings had been seismically retrofit, this work had been started with particular emphasis on those buildings categorized as essential facilities.

Loma Prieta was unusual in some respects. There was no surface fault rupture which is characteristic of earthquakes in the region, and there was unexpected heavy ground shaking at the Santa Cruz mountain tops. In many respects, Loma Prieta was all too predictable. The majority of engineered building failures occurred due to soft soil amplification of heavy ground shaking, poor maintenance, and construction deficiencies. Unreinforced masonry and nonductile concrete construction performed poorly, as expected.

One year after the event, reconstruction had barely begun in some areas, indicating that the recovery would be an extremely protracted one. Public confusion still remained regarding the purpose of building codes and the subsequent level of protection provided by a building designed and constructed to meet a specific code. Public-perceived "Government failure" to adequately protect the public against known seismic dangers has come under increased scrutiny. Reconstruction in the area has been delayed by funding problems, litigation, historic preservation interests, and code provisions regarding the seismic design requirement for the reconstruction of earthquake-damaged buildings. Loma Prieta serves as a stark reminder of the damage potential of earthquakes.

The Loma Prieta earthquake fulfilled a forecast by the US Geologic Survey (USGS). The USGS had identified the southern Santa Cruz mountain segment of the San Andreas Fault as the most likely location for a magnitude 6.5 to 7.0 earthquake during the 30-year period 1988-2018. While it is clear that much work still remains to be done in the areas of earthquake prediction and seismic studies, the fact that specific areas along faults can be identified like this indicates that the tectonics along plate boundaries are reasonably well understood.

Regional Seismic History

Intensity and magnitude

Two terms commonly used to describe an earthquake are "intensity" and "magnitude." Intensity is a subjective measure by observers of the earthquake's severity at a specific location. The United States uses the Modified Mercalli Scale, which grades observed effects into 12 categories. Magnitude does not vary with location, but rather is a measure of the absolute size of an earthquake. The most frequently mentioned magnitude scale is the Richter Scale. While the earthquake magnitudes are no longer computed as defined by Richter, the name is retained to honor one of the field's early pioneers and usually refers to the larger of the body-wave magnitude (m_b) or the surface-wave magnitude (M_s). It is important to remember that while intensity and magnitude are interdependent, there is no close correlation between the two.

Central United States

While earthquakes within continental regions are relatively rare, these intraplate earthquakes have occurred on almost every continent. Figure 1 shows some of the damaging earthquakes of the Central and Eastern United States. Intensities and magnitudes are included when known. The seismicity of the Central United States is dominated by possibly the largest shocks to have occurred in a plate interior, the New Madrid series of 1811 and 1812. Unlike most earthquakes, which consist of a single major shock followed by aftershocks, the 1811-1812 New Madrid series had four distinct, very large shocks each with accompanying aftershocks. Residents of the region were awakened after 2 on the morning of 16 December 1811 by what was to be the first of many shocks ($M_s = 8.6$). Approximately 6 hr later, the second of the large shocks ($M_s = 8.0$) struck the area. These earthquakes

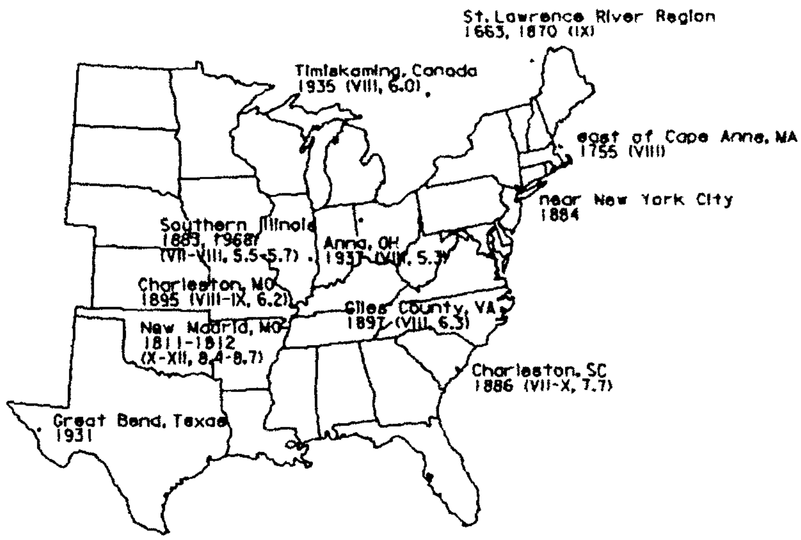
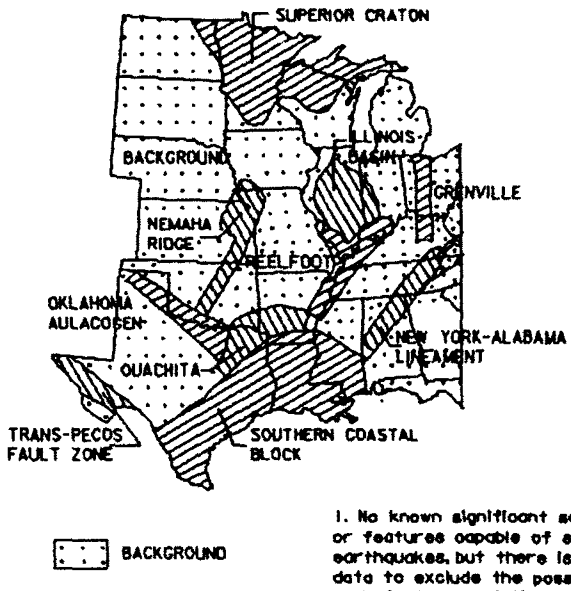


Figure 1. Typical damaging central and eastern United States earthquakes

ruptured approximately 90 miles of the southern segment of the fault. The third and fourth shocks occurred on 23 January and 7 February 1812, respectively. The third shock $M_s = 8.4$ ruptured about 45 miles of the central segment of the fault. It was the fourth and largest shock of the series ($M_s = 8.8$) that ruptured the northern branch of the fault and completely destroyed the town of New Madrid (Nuttli 1990). Records of the aftershocks were kept by Jared Brooks of Louisville, KY. By March 1812, he had recorded over 1,800 aftershocks. The aftershocks continued for at least 5 years, until 1817 (Fuller 1912). Earthquakes are known to have occurred in the New Madrid area prior to the 1811-1812 series; however, the series marks the beginning of the region's recorded seismic history. In 1843, a 6.4 moment magnitude (M) earthquake occurred near Marked Tree, AR, near the southern end of the area affected by the 1811-1812 series. This was followed in 1895 by a 6.8 moment magnitude (M) earthquake near Charleston, MO, near the northwestern end of the area affected by the 1811-1812 series (Hamilton and Johnston 1990). The region's more recent history has been punctuated by earthquakes of lower magnitude ranging from 5.0 to 5.5. The most recent area earthquake, a magnitude 4.6 with the epicenter 10 miles west of the city of New Madrid, occurred on 3 May 1991. The earthquake was felt in six states, but caused no significant damage. This type of earthquake is expected approximately every 5 years in the New Madrid Seismic Zone.

While the New Madrid Seismic Zone overshadows the Central United States, it is not the only seismically active area in the region. Figure 2 shows seismic regions of the Central United States as defined by Arch Johnston and Susan Nava (Johnston and Nava 1990).

A 12-year study funded by the Nuclear Regulatory Commission and the US Army Corps of Engineers recently completed in Kansas yielded the following seismicity information. The state experiences about two microearthquakes per month. During the period from 1987 to mid-1989, the state experienced more than a dozen perceptible earthquakes. Magnitudes as large as 5.0 to 5.5 can be expected every 100 to 200 years with earthquakes as large as 6.0 to 6.5 possible every 1,000 years. The Humbolt Fault Zone, a series of faults running through eastern Kansas from Omaha, NE, to Oklahoma City, is the



SEISMIC SOURCE ZONE	CATEGORY DESCRIPTION	EST. MAXIMUM EARTHQUAKE
Background	1	m_b 5.0-5.5
Grenville	2	m_b 5.7
Illinois Basin	3	m_b 5.7
Nemaha Ridge (Humbolt fault zone)	2	m_b 6.0
New York - Alabama Lineament	2	m_b 6.5
Oklahoma Aulacogen	4	m_b 6.8
Ouachita	4	m_b 5.8
Reelfoot Zone A - New Madrid	Zone A - 3	$m_b \geq 7.0$
Zone B	Zone B - 2	m_b 6.5
Southern Coastal Block	5	m_b < 5.0
SUPERIOR CRATON	5	m_b < 5.0
TRANS PECOS FAULT ZONE	2	m_b 6.6

1. No known significant seismicity or features capable of significant earthquakes, but there is insufficient data to exclude the possibility of such features existing.
2. Clearly defined tectonic features associated with seismicity exist, but there is no clear association with a specific fault or faults.
3. An area of greater seismicity than background (1) with clustering, but existing data is insufficient to associate the activity with crustal structures.
4. There is no evidence of significant historical seismicity, but features known to be associated with earthquakes exist.
5. No known significant seismic activity and sufficient knowledge to exclude, with high confidence, the possibility of future earthquakes.

Figure 2. Seismic source zones (Johnston and Nava 1990)

site of most of the seismic activity of eastern Kansas.

Eastern United States

The Northeastern United States is a region of relatively high seismicity principally in the St. Lawrence Valley and the Laurentian Trough. The first seismic event in most historical catalogues for the United States occurred in 1534 in the Charlevoix Seismic Zone in the St. Lawrence Valley. The zone has since experienced six large earthquakes of magnitude 6 or greater with the latest occurring in 1925. The region is characterized by small magnitude events with widespread, but not uniform, epicentral distribution and occasional larger magnitude shocks capable of causing damage (Ebel 1987).

The Southeastern United States is characterized by low-level seismic activity. There is an absence of exposed seismically active faults. The plate stress is uniform over the region in an ENE-WSW direction due to the plate-tectonic forces. The region is dominated by the Charleston earthquake ($M_s = 7.7$) of 31 August 1886. The epicenter of the earthquake was 15 miles northwest of Charleston, SC. The affected area was 5.2 million sq km with damage estimated at $5 million. Damage in Charleston was greater since it was built on man-made land which amplified ground motion. Paleoseismology techniques have been successfully used in the South Carolina area. These techniques which include radiometric-age dating of buried wood fragments have led to the identification of at least four major earthquakes prior to the Charleston event with

the oldest of these occurring more than 5,000 years ago (Snider 1990).

Central and Eastern Earthquake Characteristics

General

One of the most important factors limiting the understanding of Central and Eastern United States (CEUS) earthquakes is the lack of a widely accepted hypothesis regarding the cause of earthquakes occurring within plates. The earthquake distribution in the CEUS is highly irregular, and in most cases it is difficult to relate modern seismicity to specific geologic features except in a general way. Earthquakes in the region tend to occur in diffuse zones rather than along clearly defined fault lines. Most known faults are deeply buried by as much as 3,000 ft of poorly consolidated sedimentary rock making them harder to study. Surface fault rupture usual in western earthquakes is an extremely rare occurrence in the CEUS. Seismically active areas are commonly separated by regions with little or no seismicity.

CEUS earthquakes occur much less frequently than their western counterparts. It is this frequency of occurrence which attracts greater scientific and political interest which generates more data for study. While there are 350 years of written records for earthquakes in the east, the older records provide only limited information about the earthquakes near populated areas. The World Wide Seismic Network deployed in the early 1960's has provided the capacity to record small and moderate earthquakes. By studying these smaller earthquakes, scientists hope to better understand stress patterns and locate buried faults; however, fewer than 30 years of these records exist. Incomplete data make it impossible to prove that most CEUS seismic zones occur in a predictable pattern or manner. This makes it difficult to establish recurrence relationships. Frequency of recurrence patterns and relationships for smaller earthquakes are then extrapolated to larger magnitudes.

Earthquakes east of the Rocky Mountains are typically felt and cause damage over a much larger area than do western earthquakes. This is probably due to the homogeneous nature of the basement rock and lack of attenuation east of the Rocky Mountains.

See Figure 3 for a comparison of the areas affected by the 1906 San Francisco earthquake and the December 1811 New Madrid earthquake (Ramelli and Slemmons 1990).

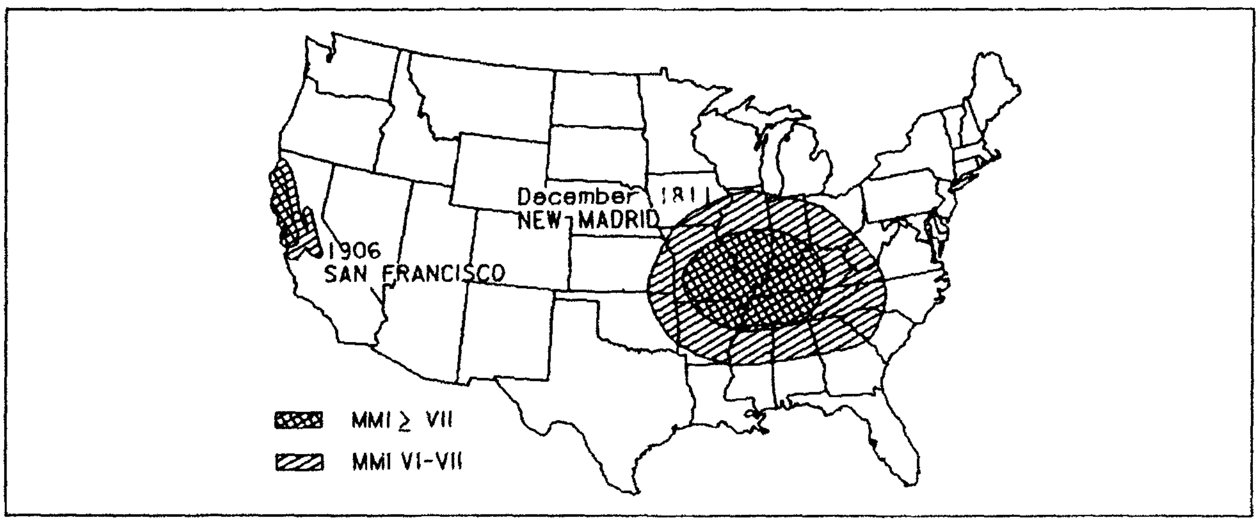


Figure 3. Areas affected by large historical earthquakes
"Lower attenuation of seismic waves in the central and eastern US result in much larger damage areas relative to the west." (Ramelli and Slemmons 1990)

Probability of occurrence

The National Center for Earthquake Engineering in Buffalo, NY, reports that there is "a very high probability that a magnitude 6 or greater earthquake will occur somewhere in the Eastern United States within the next 20 years" (Snider 1990). Studies done by Nishenko and Bollinger (1990) established the probabilities shown in Table 1. The probability for a $m_b \geq 6.0$ earthquake somewhere in the Central and Eastern United States within the next 30 years is between 40 and 60 percent (Nishenko and Bollinger 1990).

Earthquake Implications for the CEUS

General implications

A number of factors combine to make the occurrence of even a moderate seismic event potentially devastating in the CEUS.

- The Eastern and to lesser extent the Central United States have a large number of densely populated areas.

Table 1
Conditional Probability Estimates[1]

Region	1990 to 2000		1990 to 2020		1990 to 2040	
	$m_b \geq 6.0$	$m_b \geq 7.0$	$m_b \geq 6.0$	$m_b \geq 7.0$	$m_b \geq 6.0$	$m_b \geq 7.0$
New England	0.08	0.01	0.21	0.03	0.33	0.04
Southeast	0.11	0.02	0.30	0.04	0.45	0.07
New Madrid	0.13	0.02	0.34	0.05	0.50	0.08
Combined	0.29	0.04	0.64	0.11	0.81	0.18
CEUS	0.16-0.22	0.01-0.02	0.41-0.53	0.03-0.07	0.58-0.72	0.05-0.11

[1] (Nishenko and Bollinger 1990) Probabilities do not depend on the time elapsed since the last earthquake.

Table 2 shows the lack of clear agreement between the experts in the area of seismic probability. The marked difference in probabilities illustrates the uncertainty and lack of understanding of the tectonics of the New Madrid Seismic Zone. It shows the need for further research into the New Madrid Zone specifically and the CEUS in general. Establishing the probability of occurrence in an area is essential in establishing the seismic hazard for a region.

- Most local building codes lack seismic provisions. Municipalities with seismic provisions in their codes frequently address only new construction, not the problem of retrofitting existing construction.
- Building stock, particularly in older established cities, is predominantly unreinforced masonry, one of the construction types most vulnerable to seismic damage and most dangerous to occupants.

Table 2
Probability Estimates for the New Madrid Seismic Zone (Hamilton and Johnston 1990)

m_b	M_s	Probability of Recurrence	
		Next 15 years	Next 50 years
Model I - Time Dependent Model			
≥ 6.0	≥ 6.3	40% - 63%	86% - 97%
≥ 7.0	≥ 8.3	0.3% - 1.0%	2.7% - 4.0%
Model II - Time Independent Model			
≥ 6.0	≥ 6.3	16% - 24%	45% - 60%
≥ 7.0	≥ 8.3	2% - 4%	7% - 11%

- The transportation system is generally vulnerable to seismic damage, especially the bridges. This seismic vulnerability is increased by age, adverse climate in the northern regions, neglect, and poor maintenance. Bridges most vulnerable to seismic damage have one of these characteristics common to the majority of bridges in the CEUS. They are simply supported spans with deficient bearings and inadequate seat widths or have non-ductile concrete substructures or underreinforced footings or underreinforced abutment walls. Figure 4 illustrates the large number of CEUS with deficient bridges and highways (Buckle 1990).

- Numerous pipelines cross the Central United States in or near the New Madrid Seismic Zone. The 40-in.-diam Capline System, operated by Shell Oil which runs from southeastern Louisiana to Patoka, IL, is particularly vulnerable. It is possible that the pipeline could rupture during an earthquake in the New Madrid Zone and contaminate water supplies for western Tennessee (Hwang and Chen 1990).

- Surface soil near many major cities have liquefaction potential. At a minimum, these areas include New York City, NY; Charleston, SC; and the mid-Mississippi Valley. Soft soils in these areas would tend to amplify the heavy ground shaking during a seismic event.

Implications for St. Louis, MO

To better understand the seismic risk and hazard for the CEUS, it is helpful to examine a specific city as an example. St. Louis, MO, was selected for several reasons. Historically, the city has suffered damage from seismic events originating in the New Madrid Seismic Zone as well as from earthquakes originating in several other seismic areas. The tri-services Technical Manual *Seismic Design for Buildings* (Headquarters, Department of the Army 1982) classifies St. Louis as a seismic zone 2, while the 1988 edition of the *Uniform Building Code* (International Conference of Building Officials 1988) classifies the area as zone 2A. The city typifies much of the CEUS in the following ways:

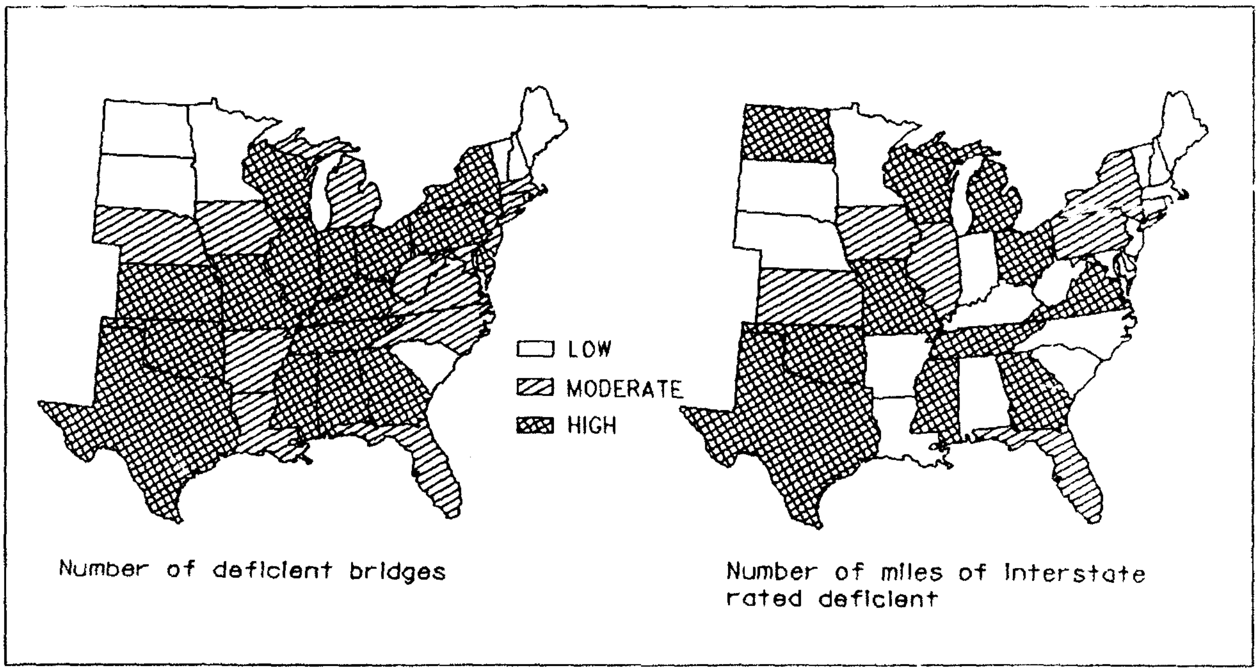


Figure 4. Deficient bridges and interstates in the central and eastern United States
(Based on information from the US Department of Transportation, 1989. *Deficient in this context is the term for substandard structural or pavement condition.) (Buckle 1990)

- The seismic risk zonation is moderate. There are few higher zoned areas in the CEUS based on current probabilistic zonation maps. The majority of the CEUS area is zone 2 or less.

- Until very recently, St. Louis had no seismic provisions in their local building codes for either the city or the surrounding county. The city and the county have recently adopted the Standard Building Code earthquake provisions for new construction. The great majority of city and county buildings are not designed for seismic loading. The adoption of seismic provisions is the exception rather than the rule in most of the CEUS.

- The city and to a lesser extent the county is predominantly unreinforced masonry construction as are most of the older urban areas in the CEUS.

Much of the following information was the result of continuing efforts of the Federal Emergency Management Agency (FEMA) and the Central United States Earthquake Consortium (FEMA 1990). The study projected the impact on St. Louis and St. Louis county of two different magnitude earthquakes ($M_s = 7.6$ and $M_s = 8.6$) in the New Madrid seismic region. To maximize the impact of the earthquake on the study area, the anticipated epicenter was located as close as possible to St. Louis. This placed the epicenter 150 miles south-southeast of the St. Louis area. Maximum ground-shaking estimates for the FEMA study were developed by Algermisson and Hopper as part of a study for the United States Geologic Survey. Figure 5 represents the maximum ground-shaking intensity for the entire region based on earthquakes' epicenters running the length of the New Madrid Seismic Zone. Figure 6 represents the upper levels of shaking likely to occur in a given region of the study area due to an earthquake of the specified magnitude. (As previously stated, this figure assumes an epicenter 150 miles south-southeast of the city.) As expected, the maximum ground shaking is expected to occur along the flood plains of the Mississippi, Missouri, and Meramac Rivers in areas of unconsolidated alluvial materials.

In an $M_s = 8.6$ event, the hypothetical maximum intensities projected range from VII to IX. As described by the Modified Mercalli Intensity Scale, the observed damage could range from negligible in well-designed and constructed buildings and considerable in building of poor design and construction for intensity VII areas to considerable damage in buildings of good design and construction with damage to underground pipes for intensity IX areas.

The projected maximum hypothetical intensities corresponding to a $M_s = 7.6$ event range from VI to VIII. The observed damage could range from slight for intensity VI areas to some damage in buildings of good design and great damage in poorly designed and constructed buildings for intensity VIII areas.

The area is particularly vulnerable because of the predominance of unreinforced masonry buildings. There are estimated to be approximately 185,000 residential unreinforced masonry buildings almost evenly divided between the city and the county. Approximately 80 percent of the city's residential buildings and 85 percent of the city's commercial and industrial buildings are unreinforced masonry construction. The percentage of unreinforced masonry in the county is much lower due to the county's larger building inventory; however, it is still a significant percentage. The seismic vulnerability of unreinforced masonry construction and the density of construction in the St. Louis area combine to ensure that most of the property damage and resulting casualties will occur in these buildings.

The study concludes that if a great ($M_s = 8.6$) earthquake were to occur (as specified), the City of St. Louis and St. Louis County could expect "thousands of casualties and property damage totalling billions of dollars" (FEMA 1990). Fire and flooding could be caused by the earthquake's contributing to damage and loss of life. Rescue and recovery efforts would be hampered by damaged facilities and

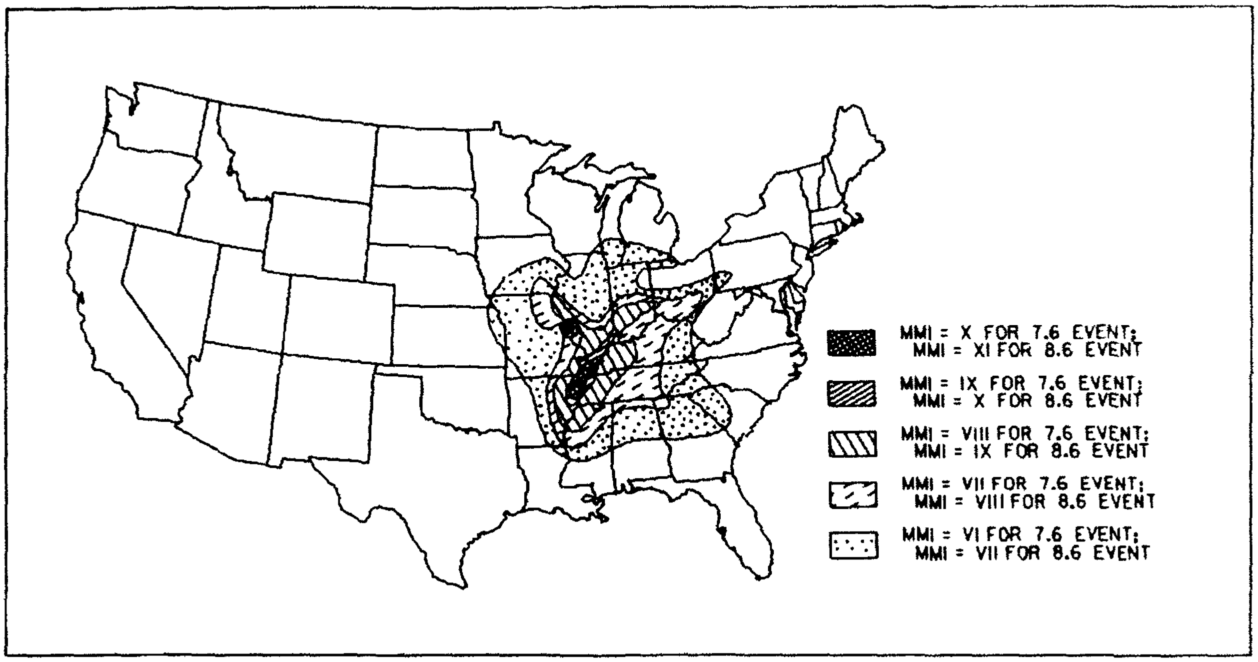


Figure 5. Map of hypothetical maximum intensities resulting from an earthquake anywhere along the New Madrid Seismic Zone (FEMA 1990)

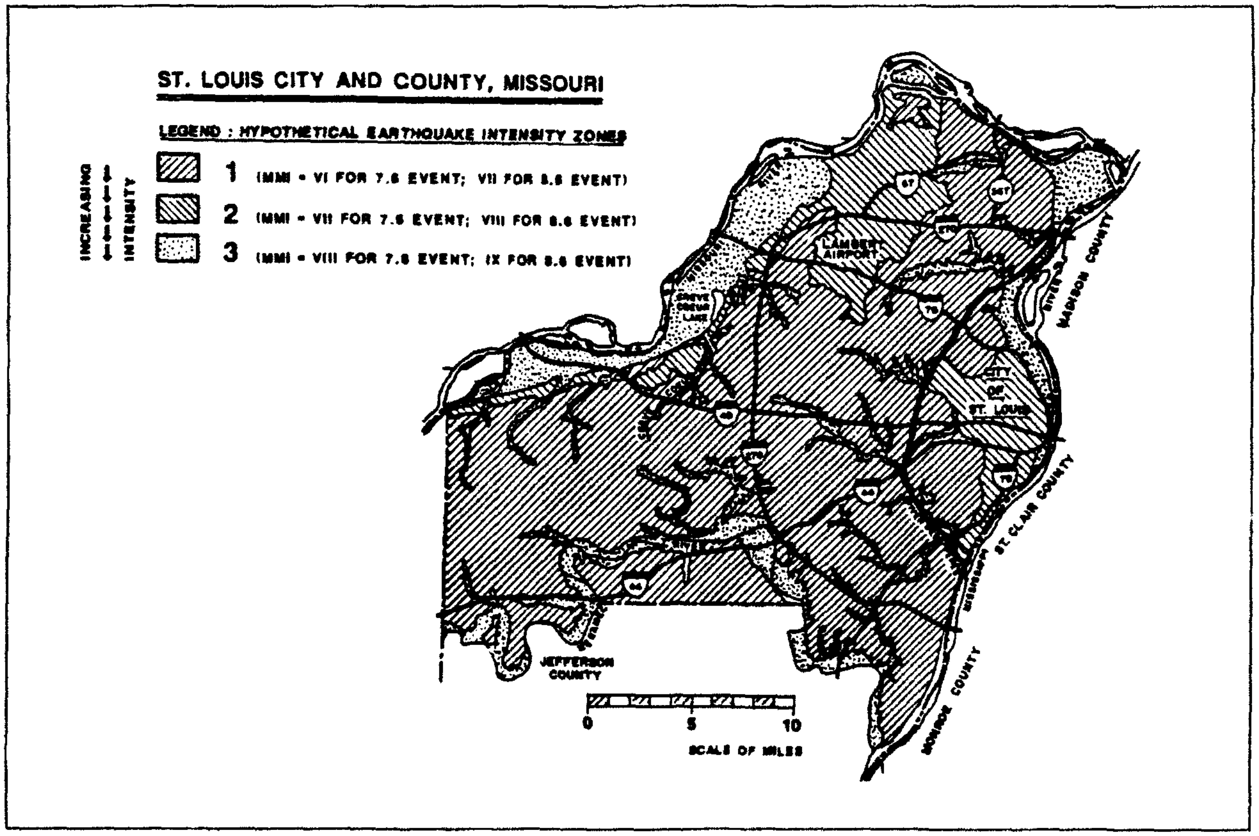


Figure 6. Map of hypothetical maximum intensities in St. Loius City and County, MO (Fema 1990)
Resulting from an earthquake in the northern part of the New Madrid Seismic Zone.

equipment, reduction of airport operational capability, and limited highway useability due primarily to damaged overpasses and bridges.

The occurrence of a major earthquake ($M_s = 7.6$) is "expected to cause 40 percent as much property damage and 20 to 25 percent as many casualties as a $M_s = 8.6$ earthquake" (FEMA 1990). All of the effects of the great magnitude earthquake are expected to a lesser extent in the smaller magnitude event.

Building codes

In general, the local political jurisdiction has the ultimate responsibility for establishing and enforcing a building code. A community can develop their own code, adopt one of the three available model codes in its entirety, or use a model code with modifications. Unfortunately, the majority of communities in the CEUS have chosen to adopt model building codes without seismic provisions. Agencies of the Federal Government are exempt from local regulations and often develop their own building regulations.

While the majority of building design and construction by the Corps of Engineers relies on the technical manuals, there is increasing pressure to be more competitive with the private sector. The use of fast track design building methods is increasing. These projects are usually based on so called "industry standards." This makes it increasingly important for us to understand the purpose and the limitations of existing building codes. As previously stated, one of the problems that became apparent following the Loma Prieta Earthquake was the lack of understanding of the purpose of building codes. Seismic codes establish minimum standards for design and construction of earthquake-resistant buildings. In general, structures should resist minor earthquakes with no damage, moderate earthquakes may cause nonstructural damage, and structures should resist major earthquakes without collapse, but both structural and nonstructural damage may occur. Codes must be simple enough so that the average practitioner can apply it correctly, at reasonable cost. The provisions must be unambiguous, with clear intent, so they are enforceable.

The increased cost for seismic design is one of the most frequently cited reasons used to justify the adoption of codes without seismic provisions. Increased cost for seismic design should be in the range of 1 to 4 percent of the cost of the structural system or 1.5 percent to less than 1 percent of the total construction cost. (This is not to be confused with the total project cost.) The increased cost of complying with seismic code provisions depends on the specific seismic code, the complexity of the structural system, the cost of the structural system in relation to the total building cost, and finally whether seismic resistance is considered in the building configuration and the materials used. Failure to consider seismic resistance early in the design or unwillingness by users or other disciplines to accommodate changes required for good seismic design can inflate the cost of seismic design (Building Seismic Safety Council 1990).

Summary and Conclusions

The seismicity of the CEUS is much greater than public perception would indicate. The CEUS have experienced strong seismic activity in the past, and such activity is expected to continue. Although experts do not agree on the method of determining probability and recurrence, they do generally concur in characterizing the probability of an earthquake ($m_b \geq 6$) occurring in the CEUS within the next 30 years as moderate to high. The mechanisms of earthquakes in the CEUS and within plates in general are not well understood, and more research is required to establish seismic hazards for various regions within the CEUS. Without clear definition of seismic hazards (including hazard areas and probability of occurrence) and the ability to address the economic concerns, it will continue to be difficult to convince politicians of the necessity to incorporate seismic design considerations in building codes.

The CEUS is vulnerable due to the population density, the lack of adequate uniform

seismic provisions for new and existing construction, the age of the infrastructure (including the concentration of unreinforced masonry buildings), and the condition of existing bridges and highways. The amount of damage from even a moderate earthquake could be substantial depending on the magnitude of the earthquake, the location of the epicenter, the time of occurrence, and how well we prepare. Loma Prieta should serve as a warning to the entire Nation of the destructive capability of earthquakes. It is up to us to heed the warning, to assess the hazards, and to minimize the risks by taking appropriate action.

References

Bolt, B. A. 1990 (Oct). "The Mechanism and Size of the San Andreas Fault Source of the 17 October 1989 Earthquake," *Putting the Pieces Together*, pp 1-11.

Buckle, I. G. 1990 (Oct). "Implications of the Loma Prieta Earthquake for the Eastern United States," *Putting the Pieces Together*, pp 65-80.

Building Seismic Safety Council. 1990. *Seismic Considerations for Communities at Risk*, Federal Emergency Management Agency Publication No. 83, pp 25-55.

Ebel, J. E. 1987 (Dec). "The Seismicity of the Northeastern United States," Report No. NCEER-87-25, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, NY, pp 178-188.

Federal Emergency Management Agency. 1990 (Jun). *Estimated Future Earthquake Losses for St. Louis City and County Missouri*, Earthquake Hazards Reduction Series 53, FEMA 192.

Fuller, M. L. 1912. *The New Madrid Earthquake*, United States Geological Survey Bulletin 494, pp 13-34.

Geotimes. 1990 (Nov). "Kansas Earthquakes Studied," Vol 35, No. 11, p 11.

Hamilton, R. M., and Johnston A. C., Ed. 1990. *Tecumseh's Prophecy: Preparing for the Next New Madrid Earthquake*, US Geological Survey Circular 1066.

Headquarters, Department of the Army. 1982 (Feb). *Seismic Design for Buildings*, TM 5-809-10, Washington, DC.

Hwang, H. H. M., and Chen, C-H. S. 1990. "Seismic Hazard Along a Crude Oil Pipeline in the Event of an 1811-1812 Type New Madrid Earthquake," Report No. NCEER-90-0006, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, NY, pp 2-1 to 2-7.

International Conference of Building Officials. 1988. *Uniform Building Code*, Whittier, CA.

Johnston, A. C., and Nava, S. J. 1987. "Seismotectonics of the Central United States," Report No. NCEER-87-0025, National Center for Earthquake Engineering Research, State University of New York at Buffalo, Buffalo, NY, pp 189-200.

________. 1990. "Seismic Hazard Assessment in the Central United States," *Neotectonics in Earthquake Evaluation*, The Geological Society of America, Inc., pp 47-58.

Nishenko, S. P., and Bollinger, G. A. 1990 (Sep). "Forecasting Damaging Earthquakes in the Central and Eastern United States," *Science*, Vol 249, pp 1412-1416.

Nuttli, O. W. 1982. "Damaging Earthquakes of the Central and Mississippi Valley," *Investigations of the New Madrid Missouri, Earthquake Region*, Geological Survey Professional Paper 1236, pp 15-20.

Nuttli, O. W. 1990 (Second Ed.). *The Effects of Earthquakes in the Central United States*, Southeast Missouri State University, pp 1-20 and 33-41.

Nuttli, O. W. 1985. "The Nature of the Earthquake Threat in St. Louis," *Societal Implications: Selected Readings*, Federal Emergency Management Agency, Publication No. FEMA-84, pp 9-1 to 9-4.

Ramelli, A. R., and Slemmons, D. B. 1990. "Implications of the Meers Fault on Seismic Potential in the Central United States," *Neotectonics in Earthquake Evaluation*, p 69.

Snider, F. G. 1990 (Nov). "Eastern U.S. Earthquakes: Assessing the Hazard," *Geotimes*, Vol 35, No. 11, pp 13-15.

The State/Federal Hazard Mitigation Team. 1990 (Jan). *State and Federal Hazard Mitigation Report for the October 17, 1989 Loma Prieta Earthquake*, pp 5-20.

US Department of Commerce. 1973. *Earthquake History of the United States*, pp 5-57.

Weber, S. F. 1985. "Cost Impact of the NEHRP Recommended Provisions on the Design and Construction of Buildings," *Societal Implications: Selected Readings*, Federal Emergency Management Agency, Publication No. FEMA-84, pp 1-1 to 1-18.

Davenport Bridge Structure No. 320
Rock Island Arsenal, Rock Island, Illinois
Detailed Fatigue Analysis

by

Donald L. Logsdon[1]

Abstract

The Iowa vehicular access to the Rock Island Arsenal and railroad Mississippi River crossing is a double deck steel truss swing span bridge built in 1896. This bridge has been in continuous service for over 90 years. Stress analyses of the bridge were performed in 1950 and 1988 for the current Cooper loadings. Repair modification and strengthening of the structure have occurred over the years and the railroad loading has changed from time-to-time. Because of its age and because of attention and concerns from bridge failures in recent times, a detailed fatigue analysis was performed on the critical members in 1989.

This paper will present the rationale and techniques used to develop the estimates for the loading history of the structure and the application of fatigue considerations contained in the American Railway Engineering Association. The resulting recommended structural modifications to the controlling members will be presented.

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Corps of Engineers Dam Safety Program

by
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(Copy of paper not available)

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Post-Tension Anchors: John H. Kerr Dam and Reservoir Roanoke River Basin, Virginia

by
Christy L. Hannan[1]

Abstract

During the 1983 Periodic Inspection of the John H. Kerr concrete gravity dam, it was recommended that an analysis be performed to determine dam stability relative to current design criteria and actual field conditions. The analysis indicated that remedial measures are required to meet the criteria. Subsequently, authorization was given in 1988 for the installation of post-tension anchors in the foundation of the dam to bring the stability of the structure into compliance with present-day engineering criteria. This paper presents the investigations, analysis, and preliminary design for the installation of post-tension anchors consistent with accepted current practice and guidance.

Dam Description

Construction of the concrete gravity dam and other related features began in May 1948 and was essentially completed in June 1951. The dam (Figure 1) is 2,785 ft long, and serves as a roadway for Virginia Primary Highway Route 4. The concrete portion consists of a nonoverflow section at the eastern abutment (600 ft), a nonoverflow intake section (571.33 ft), a 22-bay gated spillway section (1,164 ft), and a nonoverflow section at the western abutment (450 ft). Transverse contraction joints subdivide the dam into 53 independent monoliths. The dam is founded on granite gneiss rock and extends 103 ft to the spillway crest. See Table 1 for pertinent data.

Dam Stability Analysis

The primary difference between the original design assumptions and the current design criteria for gravity dam stability is how the hydrostatic uplift pressures in the foundation are treated. In the original design analysis, uplift pressure in the foundation was assumed to be 100 percent of the hydrostatic pressure acting over 50 percent of the base area. It was assumed to vary uniformly from the heel to the toe of the dam. This applied uplift pressure was not reduced to account for any relief provided by the presence of foundation drains. The current design criteria (Engineer Manual (EM) 1110-2-2200, US Army Corps of Engineers (USACE) 1958) assumes uplift at the foundation contact plane to be 100 percent of the hydrostatic pressure distributed uniformly over 100 percent of the base area. A 25-50 percent reduction in uplift can be made at the location where the drains intersect the foundation.

For this analysis all other design parameters and assumptions remain the same. This change in uplift criteria resulted in overturning instability of the spillway monoliths without sluices (monolith numbers 26-40) for the flood discharge condition where the headwater elevation

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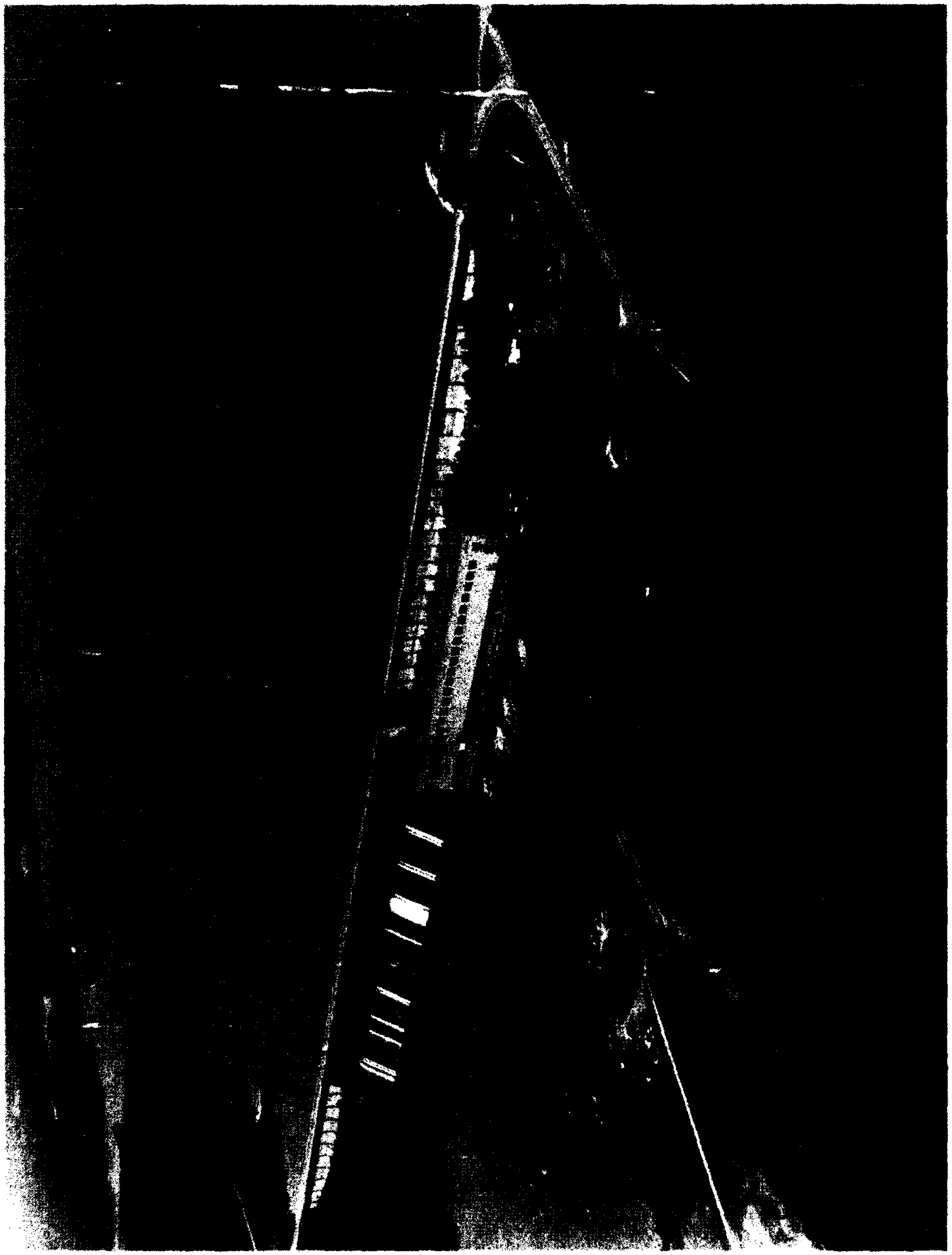

Figure 1. John H. Kerr Dam and Reservoir, Roanoke River Basin, Virginia

Table 1
Pertinent Data

Location of Damsite	
Roanoke River in Mecklenburg County, Virginia, 179 miles above mouth of river.	
Drainage Area, suqare miles	
Above John H. Kerr Dam	7,800
Entire Roanoke River Basin	9,580
Reservoir, miles	
Length (at el 320)	
Roanoke River	56
Dan River	34
Shoreline (at normal pool el 300)	800
(1) **Elevations**	
Top of surcharge pool	326
Top of flood control pool	320
Top of power pool	300
Bottom of planned power drawdown	293
Bottom of design power drawdown	268
Streambed	198
Normal tailwater	199-209
(2) **Reservoir area,[1] acres**	
Top of surcharge pool	98,200
Top of flood control pool	83,200
Top of power pool	48,900
Bottom of planned power drawdown	41,100
Bottom of design power drawdown	19,700
(3) **Storage capacity,[1] acre-feet**	
Total (el 320)	2,770,000
Flood control (el 300-320)	1,281,400
Power drawdown (el 300-268)	1,027,000
Volume at design minimum power pool (el 268)	461,600
Concrete Dam	
Length, ft	2,785.33
Elevations	
Center line roadway	332.00
Spillway	288.00
Rock/concrete interface	185.00
Spillway without sluice (monoliths 26-40)	
Monolith length, ft	50.00
Base width, ft	105.80
Drain location (downstream of heel), ft	16.50
Post-Tension Anchors (4 per monolith)	
Strands per anchor	43
Bond length, ft	35.00
Free length, ft	103.14
Post-Tension Anchors Cost Estimate	**$2,283,734**

[1] Flat pool assumption.

is 326 ft and the tailwater elevation is 209 ft (all elevations (el) in this paper are in feet referred to mean sea level).

Necessity for Rehabilitation

Based on these findings, action is being taken to compensate for the overturning potential of the spillway monoliths without sluices by developing additional moment resistance. As directed by Engineer Technical Letter (ETL) 1110-2-310 (USACE 1987), stabilizing measures are not required for the other monoliths.

Rehabilitation Plan

The most practical solution and the most common method for adding overturning resistance is the use of a post-tensioning system anchored into the foundation rock. The plan will stabilize the structure based upon current design criteria. Post-tension anchors will be installed in the overflow section without sluices at 12.5-ft centers and inclined upstream at 3 degrees from vertical. The anchors will develop an ultimate capacity of 1,500 kips each. The anchors will extend 38 ft 6 in. into the rock foundation and be approximately 141 ft 6 in. in overall length. A total of 60 anchors will be installed.

Design of Post-Tension Anchors

The post-tension anchor is composed of a high-strength steel tendon, fitted with a stressing anchorage high in the dam and a provision for load transfer through grout and into foundation rock. The rock anchor tendon is inserted into a prepared hole of suitable length and diameter, fixed to the rock, and stressed to a specified load (Post-Tensioning Institute (PTI) 1986).

Prior to design, a foundation investigation, geologic study, and core drilling to determine the quality of the rock were performed. Information from these investigations is essential in determining the type of anchor, grout, and bond length of the tendon.

Design load

The design load P is the maximum anticipated load applied to the anchor. This load is

equal to the force required by each anchor to resist the increased overturning force produced by the added uplift pressure along the base.

Based upon the monolith geometry of the spillway section without sluices and the required additional load, it is concluded that four anchors per monolith with forty-three 0.6-in.-diameter strands (1,500 kips per anchor) will be required. A 43-strand anchor requires a 10-in.-diameter installation hole. Smaller anchors could be used but more would be required. The drilling operation for the anchor holes is the most complicated and costly construction procedure in the anchor installation. Through discussions with Nicholson Construction, Bridgeville, PA (manufacturer and installer of rock anchors), it was concluded that a 43-strand tendon installation is within the capability of a qualified contractor. It is also recommended that the number of drill holes be minimized.

Bond length

The bond length is determined based upon the design load of the anchor, the diameter of the drill hole, and the working bond stress at the interface between the rock and grout. It can be estimated using the following equation as offered by the PTI (1986):

$$Lb = \frac{P}{3.14 \times d \times sw} \tag{1}$$

where

Lb = bond length

P = design load for the anchor

d = diameter of the drill hole

sw = working bond stress in the interface between rock and grout

The bond length Lb for this installation is set at 38 ft 6 in.

Anchor grout

The anchor grout will be made using Type I, II, or III portland cement conforming to American Society for Testing and Materials (ASTM) C-150 specifications (ASTM 1989). Resin has been used in the past in lieu of portland cement grout, but thorough discussions with other districts and a review of investigations by the US Army Engineer Waterways Experiment Station (Best and McDonald 1990) indicated that resin cartridges were not reliable and better results could be achieved with the portland cement grout. Tests showed that the pullout strength of resin grout placed and cured under submerged conditions is considerably less than cement grout and it exhibited significantly higher creep than the cement grout in both the wet and dry conditions. This should be considered when the frictional resistance and bond between the surfaces of the concrete and grout are important.

Corrosion protection

Due to the potential for corrosion and the desired long service life of the anchor, the steel components will be protected. Additional testing will be performed prior to completing the construction documents to better characterize the corrosive nature of the anchor environment. These tests will include electrical resistivity, pH, sulfides, and sulfates. Based on comparison of these results with critical values as determined by PTI (1986), a corrosion protection system will be designed. As a minimum, the free length will be protected with grease and sheathing with polyethylene tubing, extruded polypropylene, or other suitable material. The bond length of the tendon will be brush cleaned to ensure effective bonding between grout and tendon. Grout serves as corrosion protection and may be used for the full length as an additional means of corrosion protection.

Stressing anchorage

The stressing anchorage is the component of the installation used to transfer the prestressing force from the anchor tendon to the structure. The size and thickness of the bearing plate is determined based upon American Concrete Institute (ACI) Requirement 318-89 (ACI Committee 318 1989) and AASHTO bridge specifications (Breen, in preparation).

The required bearing plate is 26-1/2 by 26-1/2 by 4 in. thick.

The minimum size pocket of high-strength concrete required directly beneath the bearing plate is based upon EM 1110-2-2702 (USACE 1966) and Guyton (1953). The area of concrete beneath the anchorage load is subject to tensile stresses. There are two stress areas. The central portion is termed the "bursting zone" and the area along the sides and end surface is called the "tensile spalling zone." Reinforcement for these areas is determined as directed in USACE (1966) and Guyton (1953).

Construction Procedure

Work site staging

For the purpose of drilling and installing anchors, a platform may be erected on the downstream side of the spillway monoliths between the piers. The two-lane roadway across the dam may be used for onsite assemblage of the anchors, provided one lane is open at all times to traffic.

Drilling

Core drilling, rotary drilling, and percussion drilling have been used by others in establishing the required anchor holes. Core drilling is used primarily for foundation investigation with holes less than 7 in. in diameter. It is generally slower and more expensive than the other two alternatives. For these reasons, core drilling will not be used for this installation.

The rotary drilling or "down-the-hole" hammer (a type of percussion drilling) method will be used. Other types of percussion drilling will be prohibited. The contractor will determine which method he will use based on experience and ability to maintain drilling tolerances as directed by the specifications.

The contractor will be responsible for monitoring the drilling operation to ensure the hole does not veer outside a cone diverging at 1 degree from the true alignment.

The drilling equipment used will be of sufficient size to permit the penetration of miscellaneous dense material which may have been cast in the concrete or small amounts of embedded steel reinforcement.

Anchor fabrication

Tendons may be either shop fabricated or field fabricated in accordance with approved details.

Grouting

Grouting will be accomplished in accordance with PTI (1990), "Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete." Using sheathed tendons (as recommended for permanent anchors), the bond length and the free length will be grouted simultaneously. This provides a better and more economical anchor than anchors where the grout is injected in two stages. Stage grouting creates a construction joint at the top of the primary grout where this joint forms a zone for potential corrosion of the tendon.

Testing

Each drill hole will be pressure tested for watertightness. The hole will be filled with water and subjected to a pressure of 10 psi. Holes that cannot be filled with water will be considered as failing the pressure test. Such holes will be grouted, redrilled, and retested.

The first three anchors installed and a percentage of the remaining anchors will be performance tested. The performance test will be made by incrementally loading and unloading the anchor in accordance with a schedule established in the specifications. This test is used to determine (1) whether the anchor has sufficient load-carrying capacity, (2) that the free length has actually been established, and (3) the residual movement (permanent set) of the anchor. The maximum load for performance testing is 1.33P .

The remaining anchors shall be proof tested. The proof test is a fast, economical test which, when used in conjunction with performance tests, verifies anchor capacity and preloads the tendons. It is performed by incrementally loading the anchor in accordance with the specifications.

After the load is transferred to the stressing anchorage and before the hydraulic jack is removed, a lift-off reading will be recorded. The load determined from the reading will be within 5 percent of the specified lock-off load. The lock-off load is the final prestressing force in the anchor after an acceptable proof test.

Project operation restraints

John H. Kerr Reservoir may fluctuate from el 293 to 320. During typical operation, Kerr Reservoir will fluctuate near the rule curve elevations.

No more than six of the spillway gates will be inoperative at any time based upon hydraulic and operational considerations. The contractor will provide for these restrictions in his work scheduling.

A floating bulkhead for the spillway gates is available for use during the anchor installation. Upon request, the bulkhead can be installed by project personnel. However, this bulkhead can be used only during times when the lake elevation is between el 298.7 and 310.

During emergency flood conditions or the inoperability of the powerhouse generators, pool releases will be required through the tainter gates. The contractor will be required to provide a plan of emergency exit (to be approved by the Contracting Officer's Representative and the Hydraulics and Hydrology Branch, Engineering Division, US Army Engineer District, Wilmington (CESAW-EN-H)) based upon 48 hours notice of these conditions. A chart describing gate opening sequences used during emergency releases will be provided in the specifications to facilitate his work schedule and formulation of his emergency plan.

Construction Progress

Award of the construction contract is contemplated at the end of Fiscal Year 1991. It is anticipated that onsite construction activities will begin in the fall of 1992 with completion in late fall of the following year.

Construction Cost Estimate

The total estimated project cost for the post-tension anchor installation is $2,283,734.

References

ACI Committee 318. 1989. "Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary—ACI 318R-89," American Concrete Institute, Detroit, MI.

American Society for Testing and Materials. 1989. "Specification for Portland Cement," Designation C-150, *Book of ASTM Standards*, Parts 04.01, 04.02, Philadelphia, PA.

Best, J. Floyd, and McDonald, James E. 1990 (Jan). "Evaluation of Polyester Resin, Epoxy, and Cement Grouts for Embedding Reinforcing Steel Bars in Hardened Concrete," Technical Report REMR-CS-23, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Breen, John E. "Anchorage Zone Reinforcement for Post-Tensioned Concrete Girders" (in preparation), performed for American Association of State Highway and Transportation Officials and Federal Highway Administration by University of Texas at Austin, Austin, TX.

Guyton, Y. 1953. *Prestressed Concrete*, John Wiley and Sons, New York.

Post-Tensioning Institute. 1986. "Recommendations for Prestressed Rock and Soil Anchors," Phoenix, AZ.

Post-Tensioning Institute. 1990. "Recommended Practice for Grouting of Post-Tensioned Prestressed Concrete,"

Post-Tensioning Manual, Chapter 3, Phoenix, AZ.

US Army Corps of Engineers. 1966 (1 Aug). "Design of Spillway Tainter Gates," EM 1110-2-2702, US Government Printing Office, Washington, DC.

US Army Corps of Engineers. 1958 (25 Sep). "Gravity Dam Design," EM 1110-2-2200, US Government Printing Office, Washington, DC.

US Army Corps of Engineers. 1987 (17 Dec). "Stability Criteria for Existing Concrete Navigation Structures on Rock Foundations," ETL 1110-2-310, US Government Printing Office, Washington, DC.

Seismic Evaluation of the Folsom Concrete Gravity Dam

by

John S. Nickell, PE,[1] *and Dr. Robert L. Hall*[2]

Abstract

This paper discusses analyses of the seismic response of Folsom Dam, a concrete gravity dam, to a maximum credible earthquake. This dynamic response analysis estimates the maximum principal stresses in the concrete gravity section, showing where significant cracking, if any, occurs. Then, the structural stability of the main concrete portions of the dam is assessed. This leads to a determination of whether remedial measures are necessary to prevent catastrophic loss of the reservoir. These studies were conducted as a part of the Dam Safety Assurance Program.

Introduction

The Folsom Dam and Reservoir Project is a multipurpose project located in the Sacramento-San Joaquin Basin, California. The project is about 20 miles upstream from the city of Sacramento, CA, on the American River. The project, built by the Corps of Engineers during the period from 1948 to 1956, serves a drainage area of about 1,875 square miles. Seismic loading was considered during project design, but the analysis was limited to pseudostatic methods using a 0.05-g seismic coefficient with hydrodynamic forces modeled using the Westergaard parabola method. After the completion of construction, project ownership was transferred to the US Bureau of Reclamation in May 1956 for operation and maintenance.

As a part of the Dam Safety Assurance Program, the dynamic response of Folsom Dam was evaluated using a Maximum Credible Earthquake (MCE). This seismic evaluation is conducted according to ETL 1110-2-303. The dynamic response analysis provides a reliable estimate of the maximum principal tensile stresses caused by seismic loading. The computed maximum principal tensile stresses are compared against concrete material properties determined by laboratory investigation. This provides an assessment of the extent and depth of dam concrete cracking, if any. Based on that assessment, a determination was made on whether remedial measures are needed to prevent catastrophic loss of the reservoir from earthquake-induced damage to Folsom Dam. A sliding stability analysis was conducted according to ETL 1110-2-256.

Project Description

Folsom Dam, a concrete gravity dam, has twenty-eight 50-ft-wide monoliths numbered consecutively from the right abutment. The concrete gravity dam is bounded by the right wing dam and the left wing dam. Monoliths 1 through 7 interface with the right wing dam and are fully to partially embedded in the right wing envelopment fill. Monoliths 21 through 28 interface with the left wing dam and are partially to fully embedded in the left wing envelopment fill.

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The concrete gravity dam has a gross crest length of 1,400 ft, a maximum height of 343.8 ft (monoliths 14 and 15), and a crest width of about 32 ft. The crest elevation of the non-overflow section is 480.5 ft. The dam has a 392-ft-long overflow section with a crest elevation of 418.0 ft for spillway flows (monoliths 12 through 20). Spillway releases are controlled with eight tainter gates (five sized at 42 ft wide by 50 ft high; three sized at 42 ft wide by 53 ft high). Design of the overflow and nonoverflow cross sections of the dam was primarily controlled by extreme conditions of static loading.

Dam monoliths were constructed in 5-ft lifts and included the placement of a high cement content (rich) shell along the upstream and downstream faces of the monoliths from the base to the crest. A lean (low cement content) concrete was placed throughout the rest of the dam cross section. Both the concrete gravity dam and adjacent wing dams are predominantly underlain by weathered granitic rock consisting of quartz diorite of the Rocklin pluton.

Seismologic Threat

The Folsom Dam and Reservoir Project is located in the foothills near the western margin of the Sierra Nevada in central California. It is situated in a 250-mile-long by 20- to 40-mile-wide northwest-trending belt of complexly folded, faulted, and deformed metamorphic rock called the Western Metamorphic Belt. The Foothills Fault system, which includes the Melones and Bear Mountains fault zones, is contained within this belt.

Before 1975, this fault system was generally considered inactive, and the Sierran foothills assessed as an area of relatively low seismic activity. Historical earthquake knowledge did not show any damaging earthquakes in this region. Previous geologic studies did not concentrate on whether faults in the region should be considered as active and capable, in part due to seismic safety receiving only recent concern.

Then, on 1 August 1975, an earthquake measuring Richter magnitude 5.7 occurred near Oroville, CA. This earthquake, located about 60 miles north-northwest of the Folsom Dam and Reservoir Project, generated intensive investigation of the Foothills Fault system. These studies caused a reevaluation of the entire Foothills Fault system and led the State of California to declare the fault system to be active and capable of a Richter magnitude 6 to 6.5 earthquake.

Before the seismic evaluation of the Folsom Dam and Reservoir Project, a seismological study of the local area was conducted. The seismologic study concluded that the maximum credible earthquake for the Folsom Dam and Reservoir Project is an earthquake of local magnitude 6.5 on the east branch of the Bear Mountains fault zone. This fault is the closest known capable fault to the project, is in an extensional tectonic setting, and has a seismic source mechanism that is normal dip-slip.

This hypothetical maximum credible earthquake has a focal depth of about 6 miles and occurs about 8 miles away from Folsom Dam. The seismologic study concluded that this hypothetical earthquake would produce more severe shaking at the project than earthquakes originating from other known potential sources. The study also concluded that the return period for the maximum earthquake would be greater than 400 years and that reservoir-induced earthquakes caused by the project are unlikely.

Based on the seismological and geological studies, Professors Bruce A. Bolt and H. B. Seed provided two accelerograms. These accelerograms represent the horizontal ground motions that could be expected to occur at a rock outcrop from a magnitude 6.5 earthquake occurring 8 miles from the site.

	Earthquake 1 (EQ1)	Earthquake 2 (EQ2)
Peak horizontal ground acceleration	0.35g	0.35 g
Peak horizontal ground velocity	25 cm/sec	19.5 cm/sec
Bracketed duration (time > 0.05 g)	16 sec	15 sec

Vertical accelerograms generated from the horizontal components had the frequency content increased by 1.5 and the amplitudes multiplied by 0.6. The response spectra for 5-percent viscous damping were computed from the horizontal records and are compared in Figure 1. The periods of the first four mode shapes are also shown in Figure 1.

Description of Analysis

The seismic analyses of the critical cross section of the dam were conducted using the two-dimensional finite element program EAGD-84 (Fenves and Chopra 1984). This program determines the time-history response of the dam from the specified earthquake ground motions with the simultaneous effects of dam-water interaction, dam-foundation rock interaction, and reservoir bottom absorption added. Water compressibility is also included since this component can have a significant effect on the earthquake response of a concrete dam.

To ensure the accuracy of the computed dynamic response, the parameters controlling the program EAGD-84 must be judiciously selected. These parameters are chosen according to the guidelines of Fenves and Chopra (1984). Other modeling variables which impact the results were also carefully selected. All analyses used the gross, or normal maximum, reservoir pool elevation, based on hydrologic records that showed that gross pool

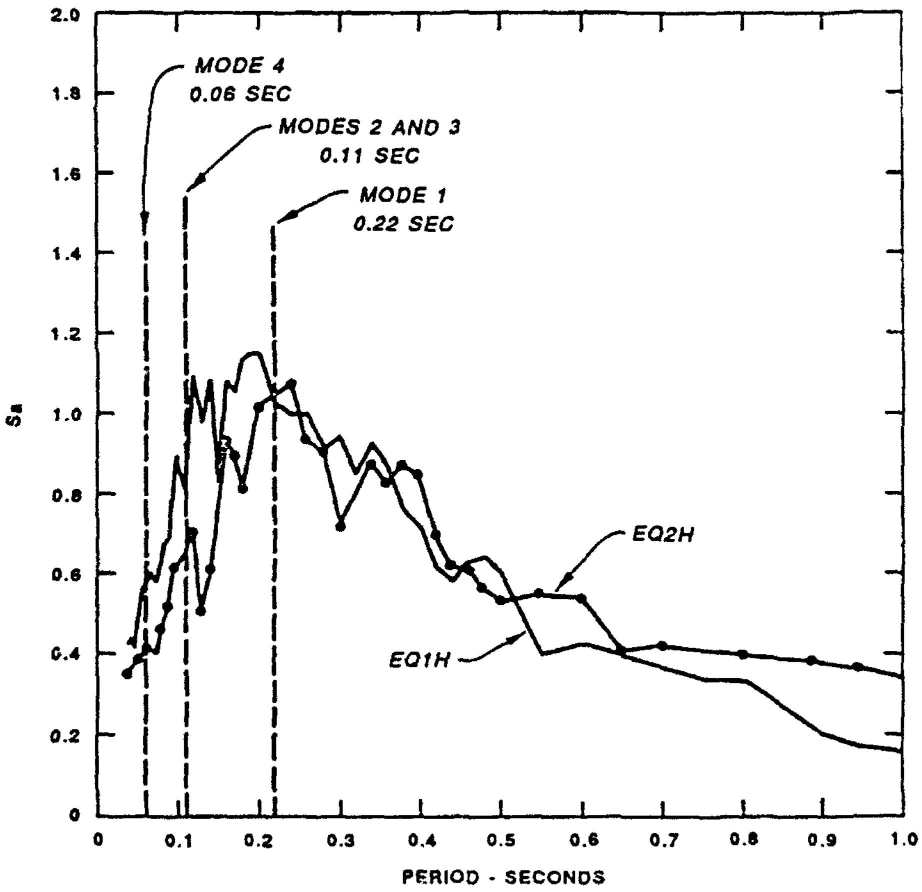

Figure 1. Horizontal response spectra for 5-percent viscous damping

was a practical worst-case reservoir elevation. Analyses were also performed to verify that the tallest nonoverflow monolith (monolith 11) is the critical cross section for evaluation. A unit weight of 158 pcf, determined by laboratory testing, was used for the unit weight of concrete in all computations. Field and laboratory investigations were conducted to determine dynamic material properties of the concrete and foundation.

Field and Laboratory Investigations

Concrete core testing was conducted on 6- and 12-in.-diam core specimens. Core holes were drilled at locations along the crest, in the interior, and on the downstream face of the dam. Both static and rapid load testing was done to define the linear-elastic properties of the dam concrete. Rapid load testing determined the concrete modulus of elasticity, Poisson's ratio, compressive strength, and splitting tensile strength. Static load tests of the modulus of elasticity and Poisson's ratio were also performed. Testing also indicated that the thickness of the rich concrete shell on the monolith face varies from 2 to 10 ft.

The critical tensile stresses in the dam are largely the result of dynamic effects from the seismic loading. Because of the nature of the loading and the limited extent of rich concrete in dam cross section, the modulus of elasticity used in the analyses is the value obtained from rapid load tests on the lean mix concrete. The recommended values shown in Table 1 were used in all analyses.

Table 1
Concrete Material Properties

Property	Value
Modulus of elasticity, dynamic	5.9×10^6 psi
Poisson's Ratio	0.19
Apparent dynamic tensile strength	
Lean concrete	700 psi
Rich concrete	840 psi

The shape of the stress-strain curve for concrete taken to failure (shown in Figure 2) shows why computed tensile stresses cannot be directly compared with the laboratory-measured splitting tensile stress. Finite element analysis is basically a strain analysis. Thus, when the deformations and forces of the finite elements are in balance, the strains everywhere are multiplied by the elastic modulus to give the stresses throughout the mass. Therefore, the tested dynamic tensile stress is increased 30 percent before comparison to the computed stress. This strength increase yields the apparent dynamic tensile strength.

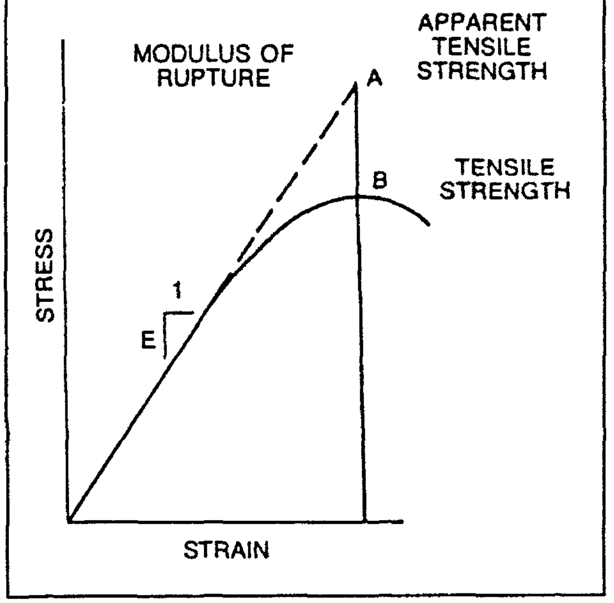

Figure 2. Typical stress-strain diagram for concrete

In situ testing of the foundation and abutment bedrock was conducted to determine the foundation rock's modulus of elasticity. Estimates for several properties of the rock beneath the Folsom Concrete Gravity Dam were prepared and are shown in Table 2. The seismic analysis used all three sets of foundation properties to assess the sensitivity of the results to the foundation stiffness.

Table 2
Foundation Rock Properties

Modulus of Elasticity, Dynamic	Poisson's Ratio	Unit Weight pcf
5.8×10^6 psi	0.30	167
7.9×10^6 psi	0.25	171
11.0×10^6 psi	0.20	174

Finite Element Model

The critical dam monolith (monolith 11) is idealized using 240 four-node nonconforming planar finite elements. This mesh (shown in Figure 3) captures the predominant modes of vibration and allows accurate evaluation of stresses throughout the monolith. The foundation rock supporting the dam is represented as a homogeneous, isotropic, viscoelastic half-plane. The total dam-foundation system is idealized as shown in Figure 3, with the ground motions input at the base of the dam.

The frequency-dependent dynamic stiffness matrix for the foundation rock is defined at the nodal points of the dam base and appears in the equations of motions for the dam. Energy dissipation in the dam alone is frequency dependent. Energy (strain) dissipation in the dam and foundation materials is represented by constant hysteretic damping. Constant hysteretic damping factors $S = 0.1$ for the dam concrete and $F = 0.1$ for the foundation rock are assumed (Fenves and Chopra 1984). The damping factor $S = 0.1$ corresponds to a 5-percent viscous damping ratio in all natural modes of vibration and is an appropriate value for the relatively large motions and high stresses experienced by the dam during strong earthquake ground motion.

The maximum excitation frequency should equal or exceed the frequencies of all of the significant harmonics in the ground acceleration record and the frequency of the highest

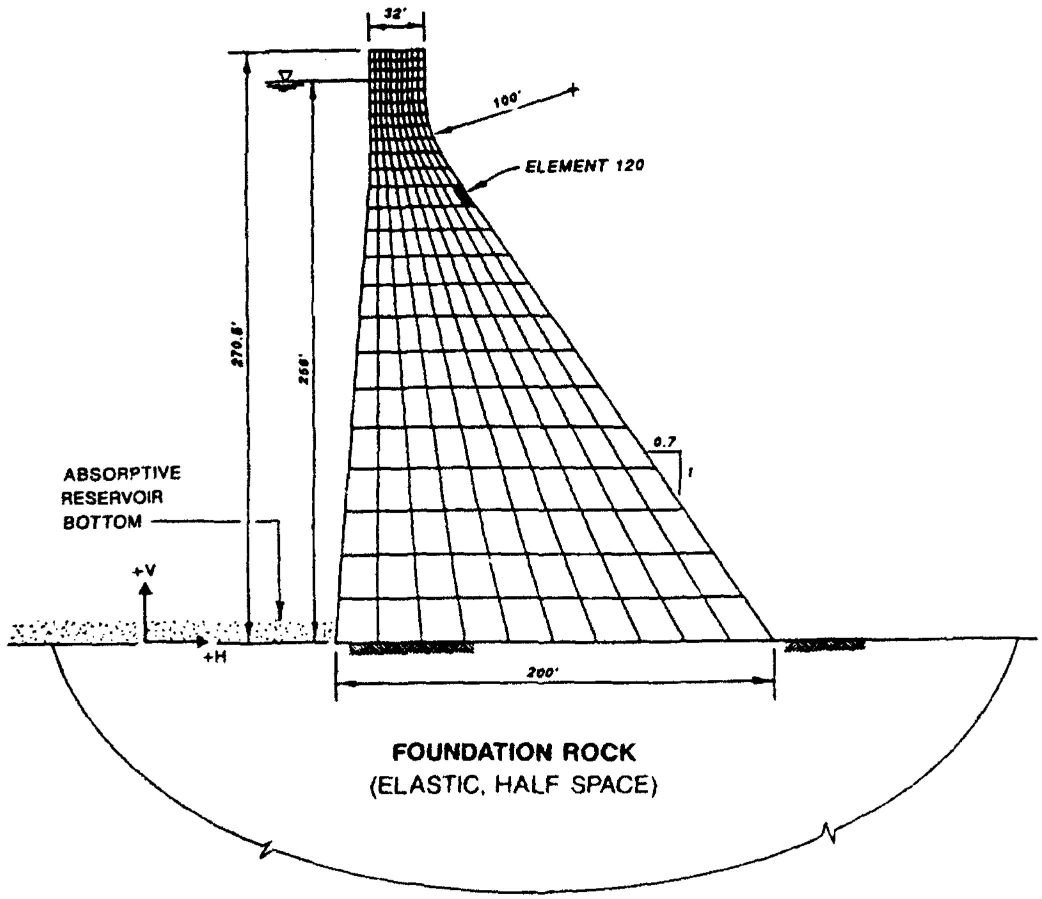

Figure 3. Total dam-foundation system with finite element mesh

mode included in the analysis. The digitized earthquake data accurately reproduce ground motion frequencies up to 25 Hz. Because the analysis includes foundation rock flexibility, 10 generalized coordinates or mode shapes are used to represent the response of the dam. The results indicate that the highest frequency of the tenth mode in any of the dam-foundation systems is 49.1 Hz; accordingly, a maximum excitation frequency of 50 Hz is appropriate.

For the specified maximum excitation frequency, the computation of the frequency response functions and the earthquake response is governed by the number of excitation frequencies and the time interval. The number of excitation frequencies used in the analysis is 1,024 (2^{10}). For a time interval of 0.01 second, which corresponds to that of the ground acceleration data, the duration of the response history is 10.24 seconds, the frequency increment is 0.049 Hz, and the maximum frequency represented is 50 Hz. The frequency increment of 0.049 Hz is less than 1/50 times the least fundamental natural frequency in any analysis, 4.4 Hz, and thus is sufficiently small to represent the frequency response functions near fundamental resonant peaks. These response parameters also satisfy the additional requirements of reducing the aliasing error in the discrete Fourier transform and ensure accurate computation of the compliance functions for the foundation rock.

The absorptive nature of the reservoir bottom is characterized by a wave reflection coefficient. This coefficient represents the dissipation of hydrodynamic pressure waves in the reservoir bottom and is modeled by a boundary condition of the reservoir bottom which partially absorbs incident hydrodynamic pressure waves (Fenves and Chopra 1984). The wave reflection coefficient is defined as the ratio of the amplitude of the reflected hydrodynamic pressure wave to the amplitude of a vertically propagating pressure wave incident on the reservoir bottom. Since the bottom materials are generally composed of variable layers of exposed rock, alluvium, and other sediments, it is generally difficult to determine reliable values of the wave reflection coefficient. Instead, using only the foundation rock modulus yielded wave reflection coefficient values of 0.75, 0.79, and 0.82, as shown in Table 3. These values are conservative because they only account for the wave absorption in the rock at the reservoir bottom, but neglect the additional wave absorption in the sediments.

Table 3
Summary of Maximum Principal Stresses

Case	Foundation Modulus	Wave Reflection Coefficient	Maximum Principal Stress
1	5.8×10^6 psi	0.75	585 psi
2	7.9×10^6 psi	0.79	672 psi
3	11.0×10^6 psi	0.82	871 psi

Stress Analysis Results

Because the monolith is nonsymmetric, stresses on the upstream and downstream faces from seismic loading will not be equal. Accordingly, earthquake forces are applied in both directions. That is, the original accelerograms were used (amplitude times +1) as well as the negative records (amplitude times -1). Thus, for each earthquake, four different sets of ground motions result: H+V, H-V, -H+V, and -H-V. A total of eight sets of ground motions result when these four ground motions are applied in both the upstream and downstream direction.

To determine which ground motion is critical, preliminary analyses were made using each foundation rock property. These analyses used an overly conservative value of 0.90 for the wave reflection coefficient and the concrete material properties shown in Table 1. The ground motion producing the highest tensile stress for the low foundation modulus was earthquake EQ1, direction H-V. For the intermediate and high moduli, the critical ground motion is EQ2, direction -H+V.

The results of the stress analyses are shown in Table 3. For each foundation rock modulus, the associated Poisson's ratio and unit weight from Table 2 are used. As shown in Table 3, the greatest principal stresses occur where the foundation modulus and reservoir bottom reflection coefficient are the largest.

For this set of parameters (Case 3), a maximum principal stress of 871 psi occurs on the downstream face at a location of 73.8 ft below the crest. This region corresponds to where the vertical downstream face transitions to an inclined surface. Element 120, identified in Figure 3, is the element which experiences this high tensile stress.

Stress Analysis Evaluation

The stress of 871 psi is greater than the recommended apparent dynamic tensile strength of 840 psi for rich concrete (Table 1). To investigate the depth to which possible cracking might penetrate, contours of envelope values of maximum principal stresses for Case 3 were prepared as shown in Figure 4. For this worst case, an area on the downstream face of only about 2 ft in depth is subjected to stresses exceeding 700 psi, the apparent dynamic tensile strength of the lean mix concrete. The maximum tensile stresses occur on the outer surface where the rich concrete, which has a higher apparent tensile strength, exists.

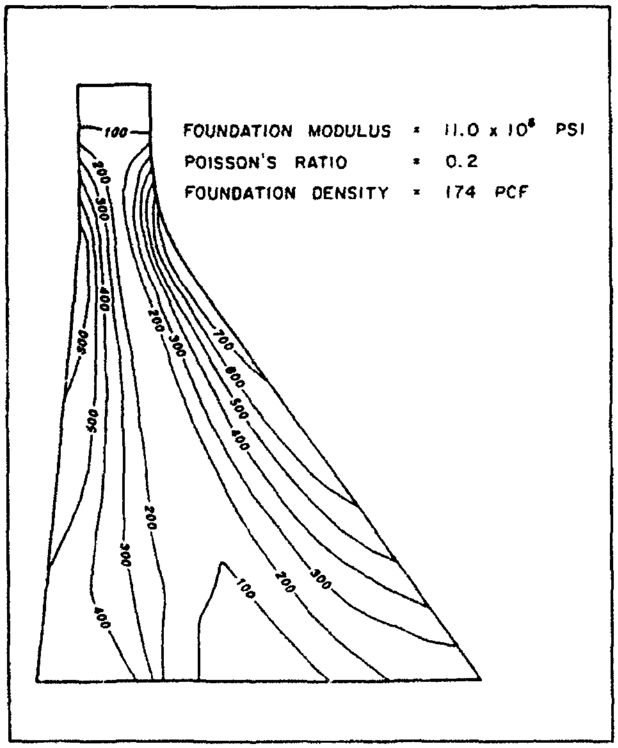

Figure 4. Envelope values of maximum principal stress

Furthermore, cracking in a concrete dam is the result of cumulative damage caused by cyclic stresses with the tensile strain exceeding the failure strain. Thus, tensile stresses (determined from linear analysis) that are greater than the maximum allowable tensile stress, and are repeated several times during an earthquake, are more damaging than a single large peak stress. In other studies of concrete dams subjected to earthquake motion, a maximum repeatable stress level is defined as the maximum stress value that is reached or exceeded by six excursions. This maximum repeatable stress level, which is considered to be more damaging than a single large transitory tensile peak stress, is compared with the apparent dynamic tensile stress. Figure 5 displays the maximum principal stress for element 120 as a function of time for Case 3. This figure indicates that the maximum tensile stress exceeds the recommended tensile strength of 700 psi only once during the entire earthquake, and that the maximum repeatable tensile stress is about 390 psi. Therefore, even though the computed stress exceeds the tensile strength of the rich concrete by 3.9 percent, it is unlikely that extensive cracking will occur.

The influence of the elevator tower, which is also located on monolith 11, on the seismic performance of the monolith was investigated as well. Increases in stress due to the presence of the tower are not large enough to change the preceding conclusion that if cracking in the monolith occurs at all, it is expected to be very limited in extent and depth of penetration. Since failure of the tower should not affect the structural stability or operation of the dam, a detailed seismic structural analysis of the monolith with the tower was not performed.

Conclusions

It is reasonable to conclude, therefore, that cracking will be quite limited in extent and depth of penetration into the monolith. It should be mentioned that in all cases, the maximum principal stresses in the region of the heel of the dam, at the upstream dam-foundation interface, were well within the tensile strength

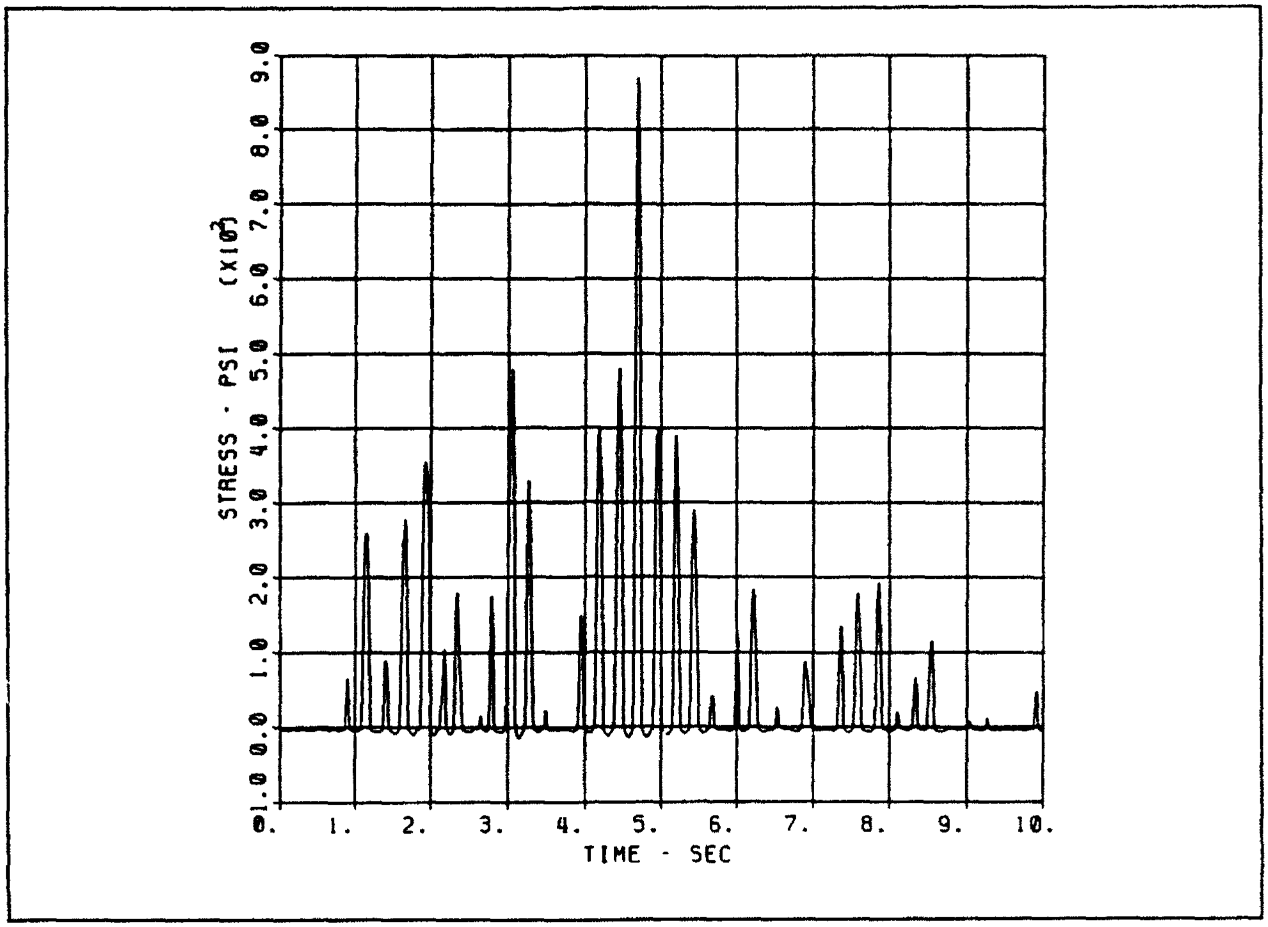

Figure 5. Maximum principal stress in element 120

limits of Table 1. Therefore, no cracking is to be expected in this location.

A stability analysis for seismic loading conditions was performed in accordance with ETL 1110-2-256. The resistance to sliding on horizontal planes at or above the nonoverflow monolith-foundation contact plane was computed. The analysis indicates that the nonoverflow concrete monolith is safe against overturning and sliding at or above the monolith-foundation contact plane.

The results show that for the parameters most likely to represent the conditions of the dam, foundation, and reservoir, little cracking will occur in the concrete portions of Folsom Dam for a seismic event. Even under the most unfavorable conditions, the analyses indicate that cracking will be limited in extent and depth of penetration. Based upon these findings, it is concluded that the dam will maintain its structural integrity during and after a major earthquake. Thus, there will be no sudden loss of the reservoir from these postulated earthquakes. Dam safety remedial measures for the Folsom Concrete Gravity Dam are not necessary.

References

Bolt, B. A., and Seed, H. B. 1983 (Sep). "Accelerogram Selection Report for Folsom Dam Project, California," conducted under Contracts No. DACW05-83-Q-0205 and DACW05-83-P-2160 for the US Army Engineer District, Sacramento, Sacramento, CA.

Cole, R. A., and Cheek, J. B. 1986 (Dec). "Seismic Analysis of Gravity Dams," Technical Report SL-86-44, US Army

Engineer Waterways Experiment Station, Vicksburg, MS.

Fenves, G., and Chopra, A. K. 1984. "EAGD-84: A Computer Program for Earthquake Analysis of Concrete Gravity Dams," Report No. UCB/EERC-84-11, Earthquake Engineering Research Center, University of California, Berkeley, CA.

________. 1986 (Jun). "Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams," Report No. UCB/EERC-85-10, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Raphael, J. M. 1984 (Mar-Apr). "Tensile Strength of Concrete," ACI Journal, pp 158-165.

________. 1986 (Sep). "Mass Concrete Tests for: Englebright Dam, Folsom Dam, and Pine Flat Dam," under Contract No. DACW05-86-P-0583, DACW05-86-P-0763, and DACW05-86-P-0855 to the US Army Engineer District, Sacramento, Sacramento, CA.

Tierra Engineering Consultants, Inc. 1983. "Geological and Seismologic Investigations of the Folsom, California Area," conducted under Contract No. DACW05-82-C-0042 for the US Army Engineer District, Sacramento, Sacramento, CA.

US Army Corps of Engineers. 1981 (Jun). "Sliding Stability for Concrete Structures," Engineering Technical Letter (ETL) 1110-2-256, Washington, DC.

________. 1985 (Aug). "Earthquake Analysis and Response of Concrete Gravity Dams," Engineer Technical Letter (ETL) 1110-2-303, Washington, DC.

Woodward-Clyde Consultants. 1983. "Investigation of Concrete Foundation Conditions at Folsom Dam: Folsom, California," Walnut Creek, CA.

Non-Linear Dynamic Analysis of the Portugues Dam

by

James G. Mangold, PE[1]

Abstract

The Portugues dam is a three-centered, double-curvature concrete arch dam scheduled to begin construction in 1992 near Ponce, Puerto Rico.

The dam's dynamic behavior was analyzed by a conventional finite element dynamic analysis, using response spectrum and time history methods. The finite element model assumed monolithic behavior by neglecting vertical contraction joint openings between monoliths due to tensile forces.

In order to get a more accurate response to dynamic loading, a non-linear dynamic analysis was performed that included vertical joint openings between monoliths. A comparison between the linear and non-linear analysis results will be presented.

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Nonlinear Response of Concrete Gravity Dams

by

Dr. Robert L. Hall[1] *and Wayne G. Johnson, PE*[1]

Abstract

The Corps of Engineers' Engineer Technical Letter (ETL) 1110-2-303 outlines a sequence of analyses of concrete gravity dams subjected to the maximum credible earthquake (MCE). The first step is a two-dimensional linear-elastic response spectrum or time-history analysis with 5 percent damping. This analysis may include the effects of hydrodynamic loads, foundation flexibility, and absorption of the reservoir bottom (Chopra 1978). For this step, 5 percent of critical viscous damping is assumed unless tensile stresses exceed 15 percent of f_c. If the tensile stresses exceed $0.15 \times f_c$, adjustments are made in the damping to account for some cracking of the concrete. These different levels of damping combined with different concrete strengths are assumed to produce a conservative and appropriate procedure.

Nonlinear dynamic calculations were performed to evaluate guidelines of the ETL. The analyses revealed that existing tools generally produce reasonable results; however, the studies demonstrated the need for further development of nonlinear analysis tools. Limitations presently exist in the modeling and assumptions of material properties (Fenves 1987), damping assumptions, and effects of water cavitation and intrusion into cracks (Dowling 1987). To ensure the safety of concrete dams subjected to strong ground motions, further research is needed on nonlinear analysis methods and the corresponding parameters that govern these complex geometric and materially nonlinear models.

Introduction

The Corps presently supports the program SDAM (Cole and Cheek 1986) and an acceleration-time-history program EAGD-84 (Fenves and Chopra 1984a,b) for the analyses of concrete gravity dams. These codes are supported through the numerical maintenance modeling program. Both codes assume linear elastic material response.

Mlakar (1986) evaluated the presently accepted procedure by performing a nonlinear analysis of three different dams of different heights subjected to two different earthquake records. The earthquake ground motions and the reservoir structure dynamic interaction were modeled with the ADINA 84 finite element code. For the cases investigated, the simpler analytic models seem to be conservative for evaluation of concrete gravity dams.

[1] Structures Laboratory, US Army Engineer Waterways Experiment Station; Vicksburg, MS.

However, the simpler procedures incorrectly located the region of maximum cracking (Mlakar 1986).

Fenves (1987) evaluated this ETL by calculating the nonlinear response of the Pine Flat Dam subjected to a scaled horizontal Taft ground motion for five different load cases. These five load cases considered a full and empty reservoir and peak acceleration varying from 0.18 to 0.45 g. The results demonstrated that cracks can form but remain stable under low amplitude ground motion; however, under large amplitude ground motion, Fenves (1987) found that the cracks propagate across the entire cross section of the studied monolith.

Detailed Studies

Mlakar's (1986) research was funded by the US Army Corps of Engineers' (USACE) Structural Research Program to evaluate the current ETL 1110-2-303. This evaluation was done by performing nonlinear analyses of three nonoverflow-gravity dam cross sections which were selected to characterize the USACE population of concrete gravity dams. The N65W and vertical components of the 1966 Parkfield, CA earthquake recorded at Temblor No. 2 Station (CIT File Nos. B037-1 and B037-3) were used for each analysis. The records from this earthquake were chosen because the peak ground acceleration and frequency content are representative of strong ground motions. The nonlinear analyses were performed using the general purpose finite element code ADINA. The constitutive behavior of the concrete was described by Bathe and Ramaswamy (1979). The hydrodynamic loading of the reservoir was modeled by adding concentrated nodal masses on the upstream face corresponding to the distribution described by Chopra (1978). All foundations were assumed rigid.

The shortest section analyzed represented the Richard B. Russell Dam on the Savannah River. This structure is 185 ft high and has a modulus of elasticity of 3,000,000 psi. The finite element grid contains 65 elements and 247 nodes providing 594 degrees of freedom. The two-dimensional cross section experienced no nonlinear behavior when subjected to the Parkfield ground motion. The ground motions were then tripled to determine the nonlinear response of this gravity dam. Figure 1 shows the cracked regions of the dam due to tripled Parkfield accelerations.

The second gravity dam modeled has a height of 300 ft, but it is not representative of any particular dam and is labeled "Standard Dam." The finite element model for the dam consists of 594 degrees of freedom. The modulus of elasticity for the mass concrete was assumed to be 3,000,000 psi. The ADINA analysis showed two cracked zones on the downstream surface at the elevation of slope change and at a slightly lower elevation (Figure 2). These cracks progress to the upstream surface. An examination of the ETL procedure for this situation reveals that no indication of cracking above the base of the dam would be realized.

The third dam analyzed was the Dworshack Dam on the Clearwater River in Idaho. This gravity dam is 638 ft high and it has an estimated modulus of elasticity of 5,000,000 million psi. This structure was chosen to represent the tallest dam owned by the Corps of Engineers. The finite element model of this dam has 80 elements and 363 nodes with 726 degrees of freedom. The nonlinear analysis indicates a cracked zone which transects the cross section of the elevation of change in downstream slope. Cracking also initiates at the upstream edge of the base and soon stabilizes (Figure 3). At later times the cracking propagates from the downstream face at the elevation of the slope change. The ETL procedure indicates that cracking is expected through the base of the structure.

Fenves performed a nonlinear seismic analysis of the tallest (400 ft) nonoverflow monolith of the Pine Flat Dam. The finite element model of the structure consists of 162 nodes comprising 136 quadrilateral elements with a total of 315 degrees of freedom. The finite element model of the impounded reservoir water extends upstream 1,200 ft from the dam

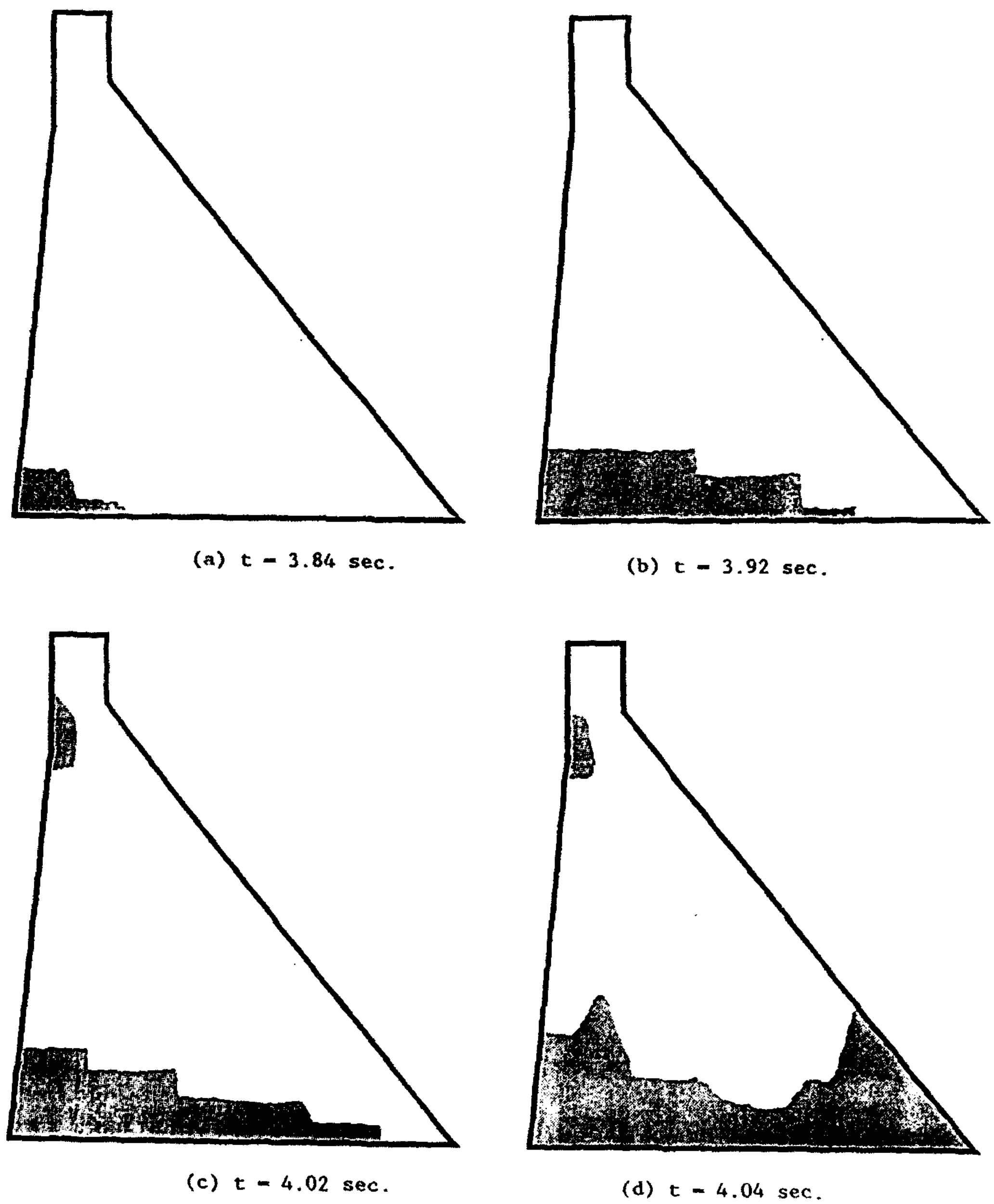

Figure 1. Cracked zones of Russell Dam with tripled Parkfield loading

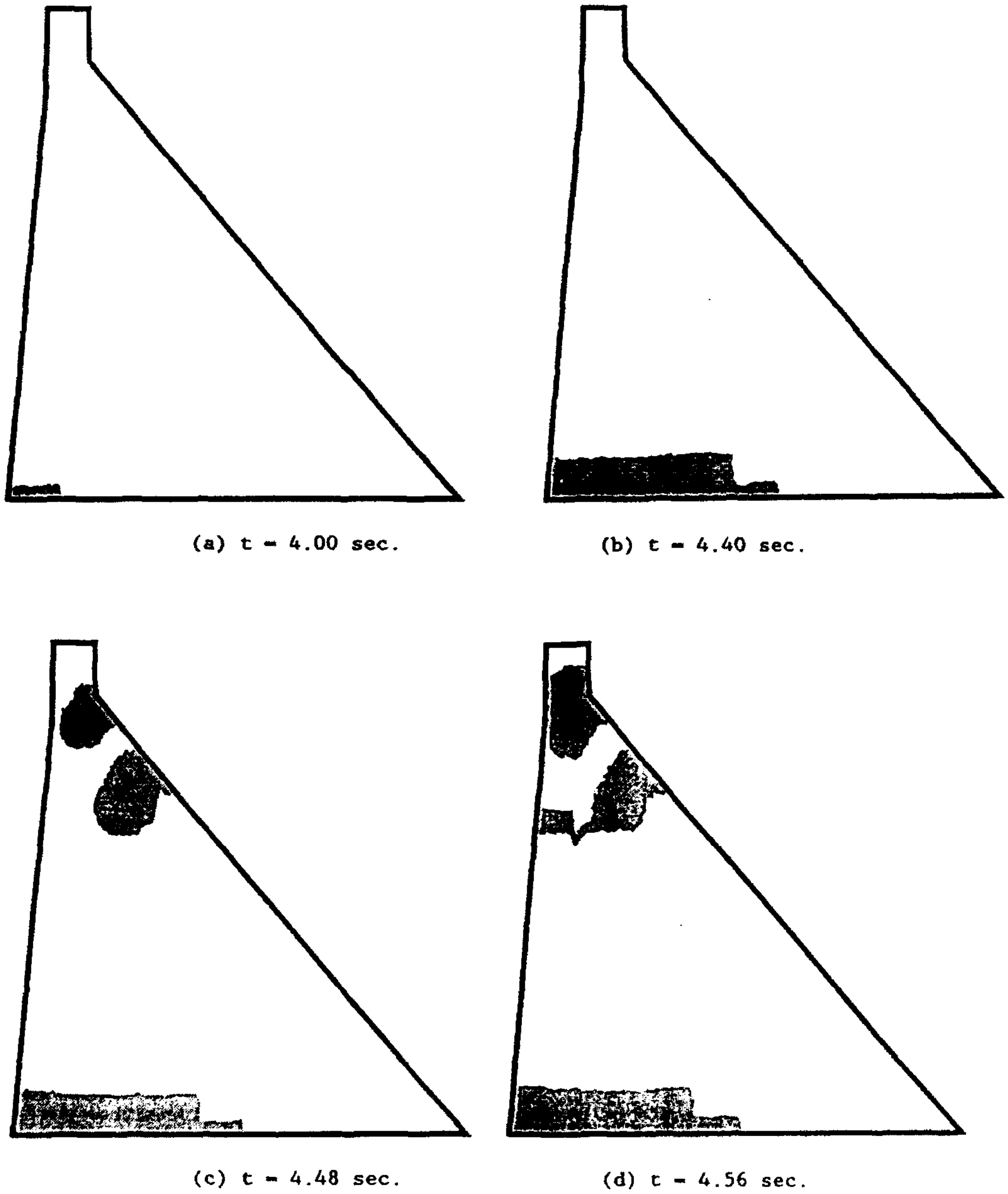

Figure 2. Cracked zones of Standard Dam

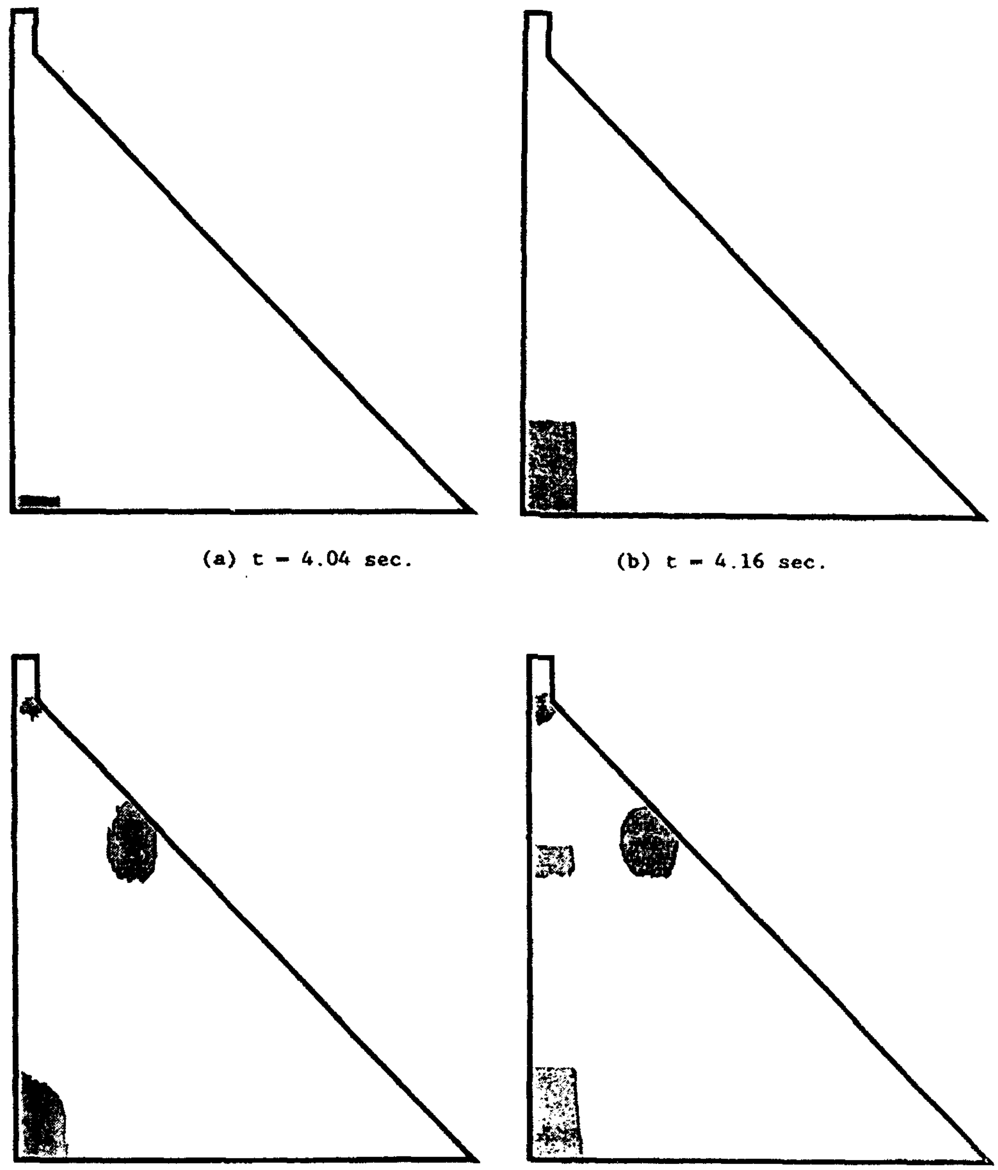

Figure 3. Cracked zones of Dworshak Dam

and consists of 224 constant pressure elements. By modeling the reservoir, the analysis accounts for the interaction between the reservoir and dam. Stiffness proportional damping was used to provide 5 percent critical damping of the fundamental frequency and a modulus of elasticity of 3.25 million psi was used. The S69E component of the 1952 Taft ground motion was taken as a horizontal component acting in the upstream-downstream direction. The peak ground acceleration from these ground motions was 0.18 g.

When the reservoir was empty, the Taft ground motion had to be scaled up by a factor of 2.5 to an acceleration of 0.45 g to initiate cracking. However, when the reservoir was assumed full (381 ft), the nonlinear analysis indicates cracking from unscaled Taft ground motions. The cracking occurs at the head of the dam where a stress concentration exists because of the assumed rigid foundation. When ground motions were scaled to 0.27 g, cracks are shown to propagate further along the base. When ground motions were scaled to 0.36 g, cracking also occurs on the downstream slope where the change in the slope occurs. The extensive cracking led to a numerically unstable solution which indicates that stability analyses are needed. Figure 4 gives a summary of these results.

The ETL procedure was followed using the computer program EAGD-84. Five load cases were investigated for the Pine Flat Dam. Table 1 summarizes the five cases and results. For cases 1, 3, and 4 the maximum tensile stresses were less than 638 psi (0.15 f'_c), so no cracking of the monolith was presumed. Cases 2 and 5 were performed again assuming 7-percent viscous damping which equates to 14-percent hysteretic damping (Table 2). Case 2 resulted in maximum tensile stress of 563 psi at the downstream face which exceeds $0.10 \times f'_c$ and, according to the ETL, a crack should be assumed at two locations and sliding stability analyses performed for the portion of the dam above this plane. For Case 5, the $0.10 \times f'_c$ criterion was exceeded on the upstream and downstream faces as well as at the heel. The ETL criteria again require stability analysis to be performed.

These nonlinear analyses clearly indicate the importance of the nonlinear response of the Pine Flat Dam to these ground motions. The ETL criteria appeared to be reasonably accurate for these five analyses. However, the procedure needs further investigation because there is no theoretical basis for assuming that tensile cracking results in increased energy dissipation which is implied by greater damping ratios.

Nonlinear Research Needs

The Panel on Earthquake Engineering for Concrete Dams Committee, Division of Hazard Mitigation, through the National Research Council, has recently completed a publication entitled, "Earthquake Engineering for Concrete Dams: Design, Performance, and Research Needs" (NRC 1990). This publication presents details for needed research in the nonlinear seismic response of concrete dams. The following are the items which should be addressed in future research programs.

Material testing of mass concrete

Further testing of mass concrete under dynamic loads is needed. It is needed to determine tensile cracking of the mass concrete under multiaxial stress states which represent the in situ strain paths in a concrete dam during a seismic event. These tests must quantify the effects of strain rates, concrete mixtures, and aggregate size. Concrete samples should be a mixture of cores from actual dams and carefully prepared laboratory samples.

Development of materials models for concrete

Once the test data are available, realistic numerical models for tensile cracking under dynamic loads can be developed. These models must allow for multiaxial stress states, strain rate effects, shear transfer by aggregate interlock, and criteria for tensile cracking and propagation of cracks. The studies must include smeared-crack approach, fracture mechanics principles, cracking-consistent damping, and the discrete-crack approach.

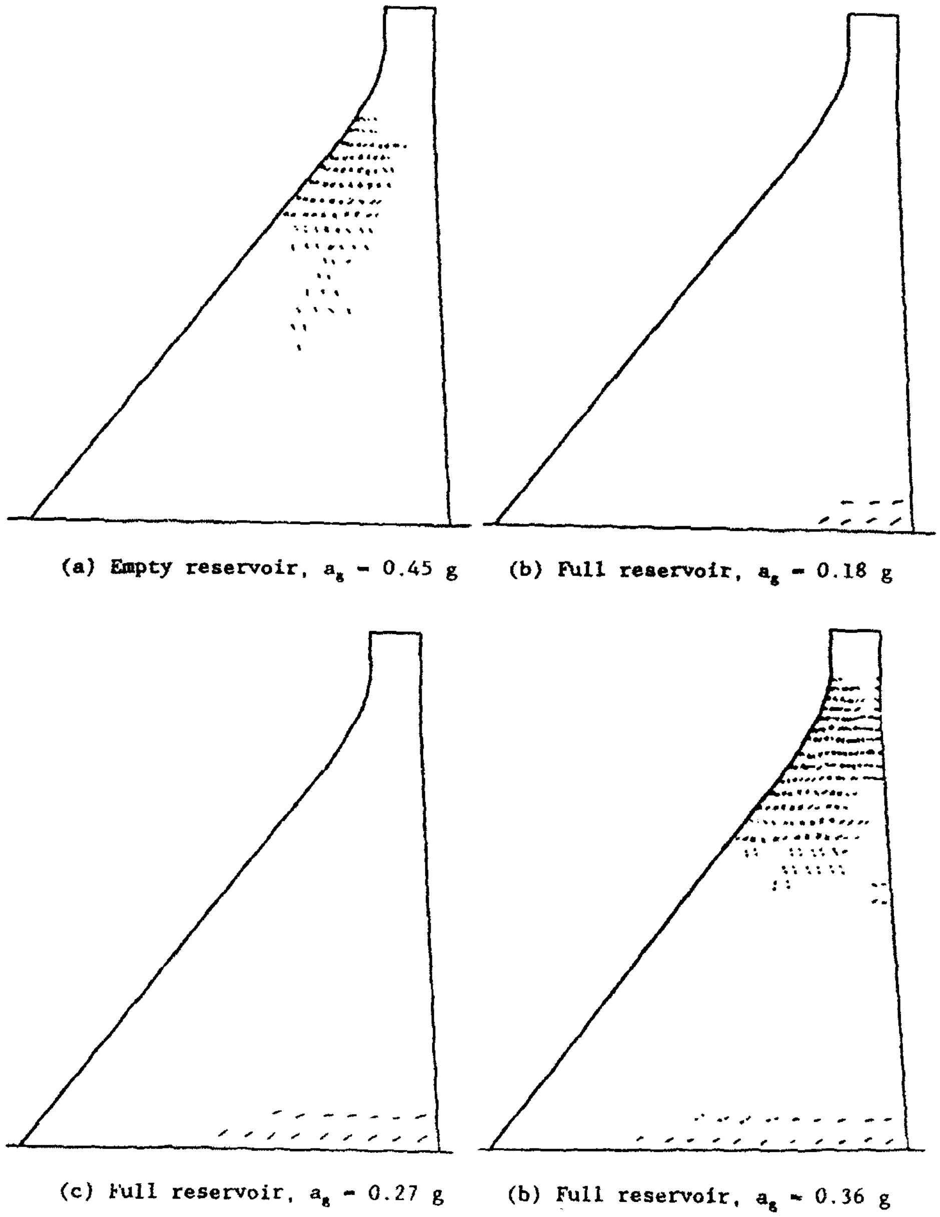

Figure 4. Tensile cracks in Pine Flat Dam due to Taft ground motion scaled to peak ground acceleration, a_g

Table 1
Maximum Tensile Stress in Pine Flat Dam Due to Scaled Horizontal Taft Ground Motion from First Linear Dynamic Analysis

Case (1)	a_g			Max. Stress (psi)		
	Water (2)	(g) (3)	μ_f (4)	Up (5)	Down (6)	Heel (7)
1	Empty	0.18	0.10	130	256	4
2	Empty	0.45	0.10	466	666	271
3	Full	0.18	0.10	217	250	361
4	Full	0.27	0.10	415	363	539
5	Full	0.36	0.10	510	580	718

Table 2
Maximum Tensile Stress in Pine Flat Dam Due to Scaled Horizontal Taft Ground Motion from Second Linear Dynamic Analysis

Case (1)	a_g			Max. Stress (psi)		
	Water (2)	(g) (3)	μ_f (4)	Up (5)	Down (6)	Heel (7)
1	Empty	0.18	—	—	—	—
2	Empty	0.45	0.14	383	563	198
3	Full	0.18	—	—	—	—
4	Full	0.27	—	—	—	—
5	Full	0.36	0.14	459	479	642

Modeling of other nonlinear mechanisms

Since the limiting tensile strength of the concrete is across lift surfaces, it is important to develop construction joint models. These models will redistribute the forces across a joint as the two surfaces open and close. The degradation of the concrete across the joints must be modeled due to the damaging number of loading cycles from an earthquake.

Numerical procedures for computing nonlinear response

The numerical material and joint models can then be incorporated into a finite element program with time integration for the equations of motion or other discretization methods for solving dynamic nonlinear equations. The procedures must also include interaction with the impounded water, flexible foundation rock, and the reservoir bottom absorption. These procedures must be refined to take advantage of vector and parallel processes in the latest computers.

Parameters and detailed response studies

With the development of accurate numerical procedures, parametric studies can be performed to determine the sensitivity of the nonlinear response to the parameter describing the nonlinear models. These studies would identify the significance of tensile cracking and joint opening with respect to failure or dynamic response. Finally the following factors should be determined: effects of ground motion characteristics, water compressibility, foundation flexibility, reservoir bottom absorption, and modeling issues.

Dynamic testing of dam models

Testing of dam models is essential for verifying nonlinear numerical procedures. These tests may require association with international augmentation testing and use of facilities such as a large earthquake simulator recently installed at the research laboratory

of the Ministry for Water Conservancy and Hydroelectric Power in Beijing, China.

Identification of design criteria

Accurate design criteria based on numerical and experimental studies could then be developed. Nonlinear seismic analysis may not become a standard practice in design but will certainly define the tensile strength of the structure and the post cracking stability.

Investigation of earthquake-resistant design measures

These nonlinear capabilities will allow for the investigation of innovative measures for increasing the seismic safety of concrete gravity dams. These tools could be used to study effects in the geometry of dams, jointing schemes, and joint materials to dissipate energy.

Conclusion

Nonlinear capabilities are important in determining the seismic stability of concrete gravity dams. The capability to develop a complete program as described is beyond single capability of the Corps of Engineers. However, the Corps can strategically utilize the research being performed through the National Science Foundation and other organizations to answer these complex issues and develop the necessary tools and criteria. Nonlinear analysis will provide the necessary insight for the development of reliable cost saving earthquake resistant designs.

References

Bathe, K. J., and Ramaswamy, Seshadri. 1979. "On Three-Dimensional Nonlinear Analysis of Concrete Structures," *Nuclear Engineering and Design*, Vol 52, No. 3, pp 385-409.

Chopra, A. K. 1978. "Earthquake Resistant Design of Concrete Gravity Dams," *ASCE Journal of the Structural Division*, American Society of Civil Engineers, Vol 104, No. ST6, pp 953-971.

Cole, R. A., and Cheek, J. B. 1986 (Dec). "Seismic Analysis of Gravity Dams," Technical Report SL-86-44, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Dowling, M. J. 1987. "Nonlinear Seismic Analysis of Arch Dams," Report No. EERL 87-03, Pasadena, CA.

Fenves, G. 1987. "Earthquake Induced Cracking in Concrete Gravity Dams," American Society of Civil Engineers.

Fenves, G., and Chopra, A. K. 1984a. "EAGD-84, A Computer Program for Earthquake Analysis of Concrete Gravity Dams," Report No. UCB/EERC-84/11, Earthquake Engineering Research Center, University of California, Berkely, CA.

Fenves, G., and Chopra, A. K. 1984b. "Earthquake Analysis and Response of Concrete Gravity Dams," Report No. UCB/EERC-84/10, Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, CA.

Mlakar, P. F. 1986. "Nonlinear Response of Concrete Gravity Dams to Strong Earthquake-Induced Ground Motion," JAYCOR Report No. J650-86-002/1335, for the US Army Engineer Waterways Experiment Station, under Contract No. DACW39-85-M-4964.

National Research Council. 1990. *Earthquake Engineering for Concrete Dams: Design, Performance, and Research Needs*, National Academy Press, Washington, DC.

US Army Corps of Engineers, "Earthquake Analysis and Design of Concrete Gravity Dams," Engineer Technical Letter No. 1110-2-303, Washington, DC.

Seismic Evaluation of Intake Towers

by
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Abstract

At Corps of Engineers civil works projects, intake towers are recognized as the structural feature most at risk to earthquake hazards. Accordingly, the Corps has devoted considerable effort in recent years to assimilate and develop seismic criteria for assessing the adequacy of reinforced concrete towers. The updated criteria will be published in Engineer Technical Letter 1110-8-8(FR), "Seismic Design and Evaluation of Intake Towers" (Headquarters, Department of the Army, in preparation). This paper presents results of an evaluation of the existing intake towers at Edward MacDowell Dam and Otter Brook Lake, New Hampshire, using the cantilever beam response spectrum method recommended in the current criteria. Based on these results, findings which are generally applicable to seismic evaluations of all existing intake towers are discussed.

Introduction

Background

The US Army Engineer Division, New England, operates and maintains 15 projects which have a free standing or partially embedded tower as one of the structural features. Since most of these towers were constructed in the 1940's through 1960's, their seismic design was based on the traditional seismic coefficient method as found in Engineer Manual 1110-2-2400. The Division is currently in the process of evaluating these towers using the current dynamic analysis procedures and criteria presented in Engineer Technical Letter (ETL) 1110-8-8(FR), "Seismic Design and Evaluation of Intake Towers" (Headquarters, Department of the Army, in preparation). To date, the towers at Edward MacDowell Dam and Otter Brook Lake, New Hampshire, depicted in Figures 1 and 2, have been investigated. The results from these two evaluations will be used to make a preliminary assessment of all towers within the Division's jurisdiction and will form the basis for future investigations.

1982 Gaza Earthquake

On 18 January 1982, an earthquake of Richter magnitude 4.7 struck central New Hampshire. This earthquake, which was centered about 1.5 miles southwest of the village of Gaza, was felt over most of New England and New York. Strong motion records were registered at five Corps of Engineers projects. Franklin Falls Dam, located approximately 6 miles from the epicenter, registered a peak acceleration of 0.55g which is the highest value recorded east of the Rocky Mountains (Krinitzsky and Dunbar 1986). This event provided the impetus for additional seismological studies and structural evaluations in New England.

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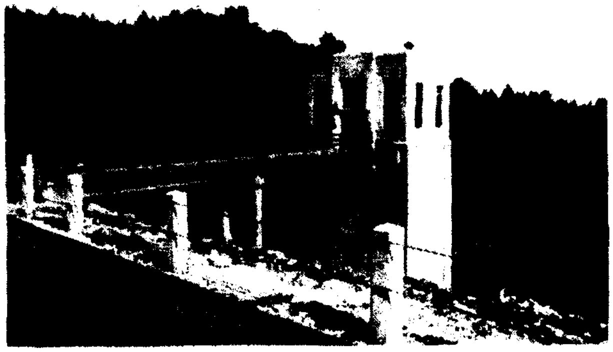

Figure 1. Intake tower at Edward MacDowell Dam

Figure 2. Intake tower at Otter Brook Lake

Overview of existing intake towers

New England Division's inventory includes towers with square, rectangular, circular, and octagonal cross sections. Tower heights vary between approximately 80 and 288 ft. Considered as representative of the towers in the inventory and suitable for evaluation by the cantilever beam response spectrum method, the towers at Edward MacDowell Dam and Otter Brook Lake were selected as the first two in the inventory to be investigated. The tower at Edward MacDowell Dam, completed in 1950, is 86.5 ft high and rectangular in cross section. The tower at Otter Brook Lake, completed in 1958, is 138.5 ft high and octagonal in cross section. In the original design computations, the towers at Edward MacDowell Dam and Otter Brook Lake were designed with a seismic coefficient of 0.10g and 0.05g, respectively.

Seismic Evaluation

Seismic load information

The first step in a seismic evaluation is the selection of appropriate earthquake data. Two levels of earthquake are prescribed in the current criteria. Generally, intake towers are to be evaluated to resist a design level earthquake, which is defined as the earthquake generated from a specific seismic source which produces ground motions at the site that have a 10-percent chance of exceedance in 100 years. In special cases where failure of the tower due to an earthquake can lead to failure of the dam and cause loss of life, towers are to be evaluated for a maximum credible earthquake, which is defined as the earthquake generated at a specific seismic source which produces the largest ground motion at the site.

Current guidance indicates that seismic load information can be obtained as either site-specific or nonsite-specific data. Nonsite-specific response spectra for a design level earthquake are contained in ETL 1110-8-8(FR) for use where site-specific spectra do not already exist. The nonsite-specific response spectra can be scaled using the effective peak ground acceleration prescribed in the current ETL and a seismic zone map, which is adopted from the *Uniform Building Code* (International Conference on Building Officials 1988), to adjust for the location of the site. Nonsite-specific response spectra were used for the evaluation of the towers at Edward MacDowell Dam and Otter Brook Lake.

One of the shortcomings of the traditional seismic coefficient method is the lack of accuracy in specification of the seismic coefficient, since the same value is assigned to a large geographic area and does not account for the seismicity of a particular location. It is noted that the same shortcoming is inherent when using the nonsite-specific data contained in the current guidance. This nonsite-specific data indicates that all of the towers in New England Division's inventory are located in seismic zone 2A for which an effective peak ground acceleration of 0.20g is prescribed.

As an example, the effective peak ground accelerations at four New England Division projects for which site-specific geological-seismological investigations have been performed are contained in Table 1. A review of these accelerations indicates the variation which can occur within a seismic zone.

Table 1
Effective Peak Ground Accelerations (EPGA)

Project[1]	Site-Specific EPGA for Design Level Earthquake	Site-Specific EPGA for Maximum Credible Earthquake
Franklin Falls Dam, Franklin, NH	Not Available	0.38g
Knightville Dam, Huntington, MA	0.25g	0.25g
Surry Mountain Lake, Keene, NH	≤0.16g	0.16g
West Thompson Lake, Thompson, CT	≤0.16g	0.16g

[1] *All projects are located in seismic zone 2A for which a nonsite-specific EPGA of 0.20g is prescribed for a design level earthquake per current criteria.*

Analysis Techniques

Both simple and complex analysis techniques are discussed in the current guidance. For towers which are generally square, rectangular, or circular in plan, the cantilever beam response spectrum method, the simplest of the acceptable techniques, is recommended. For towers with irregular cross sections or other discontinuities, a more complex finite element model is necessary. The cantilever beam response spectrum method can be readily performed using a simplified two-mode added-mass method (Chopra 1981). This method uses the first two modes of vibration and the added-mass concept, which accounts for hydrodynamic effects, to determine internal shears and moments. Computations can be performed using hand calculations or spreadsheet software. A cantilever beam response spectrum analysis by the simplified two-mode added-mass method was deemed appropriate for evaluating the towers at Edward MadDowell Dam and Otter Brook Lake.

For both towers, a nonsite-specific 5-percent damped response spectrum with an effective peak ground acceleration of 0.20g was used in the analysis. The internal moments obtained from an analysis of each tower in the upstream-downstream direction are presented graphically in Figures 3 and 4. In addition, internal moments obtained by the traditional seismic coefficient method for a 0.20g acceleration are shown for comparison.

Using the same value of effective peak ground acceleration in both methods of analysis, it is observed in Figures 3 and 4 that the results obtained by the traditional seismic coefficient method are nonconservative when compared with results obtained by the current cantilever beam response spectrum method. The degree of nonconservatism is actually greater than depicted given that the original design of the towers at Edward MacDowell Dam and Otter Brook Lake was based on an acceleration of 0.10g and 0.05g, respectively. Extrapolating from these observations, it appears that for all towers in the Division's inventory, the forces generated during a design level earthquake would exceed the seismic forces for which the towers were originally designed.

Capacity versus demand

To satisfy current criteria, intake towers must have sufficient capacity to resist the demand of a design level earthquake or, in special cases as previously discussed, the maximum credible earthquake. The capacity of the tower is considered to be the ultimate strength computed in accordance with the requirements of the American Concrete Institute (1989), while the demand, or required strength, is calculated using a load combination presented in the current criteria. The required load combination is shown as Equation 1.

$$U = 3/4\,(1.4D + 1.7L + 1.3\,E/u) \qquad (1)$$

where

U = required strength to resist factored load

D = dead loads, or related internal moments and forces

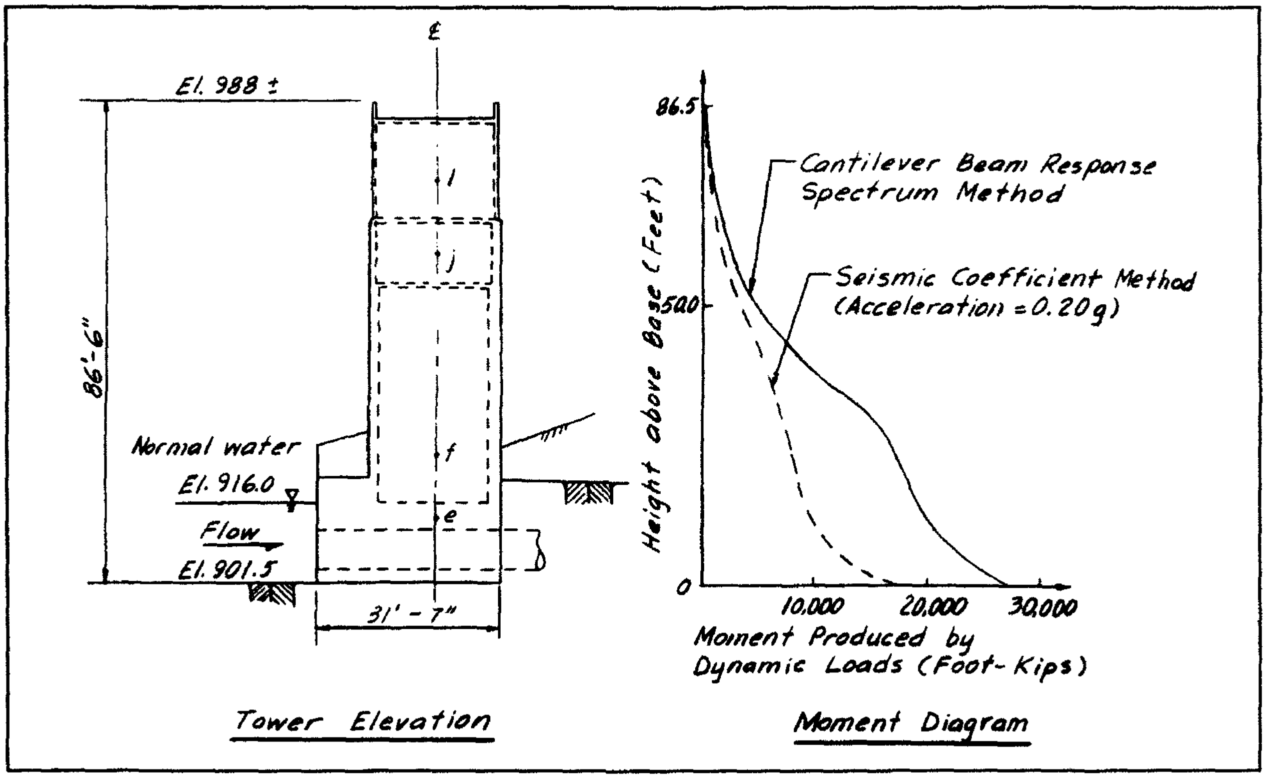

Figure 3. Moment diagram for Edward MacDowell Dam

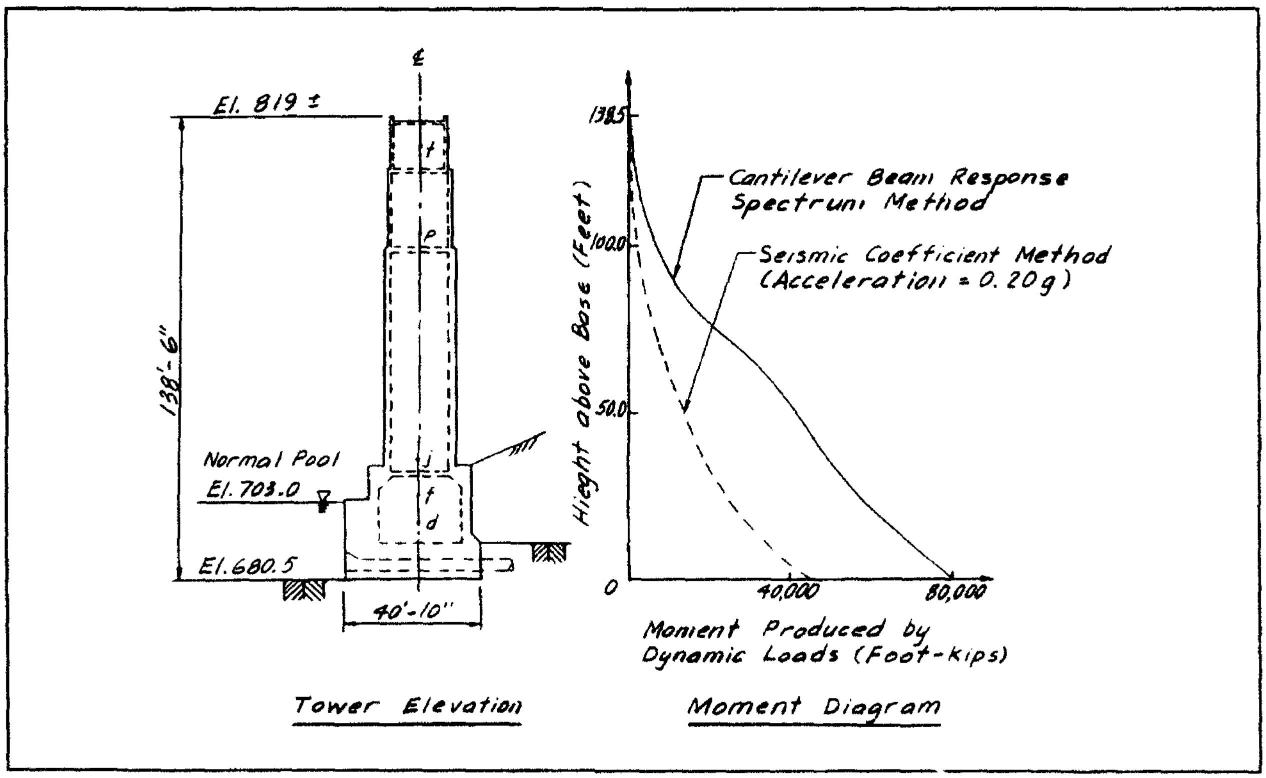

Figure 4. Moment diagram for Otter Brook Lake

L = live loads, or related internal moments and forces

E = earthquake loads, or related internal moments and forces

u = ductility factor: 2.0, if ductility requirements in current criteria are satisfied; 1.0, otherwise.

In Equation 1, it is noted that the load factor of 1.3 for earthquake loads and use of a ductility factor to reduce the earthquake loads are significantly different from the requirements of the American Concrete Institute which prescribe a load factor of 1.87 and make no allowance for ductility. The load combination in ETL 1110-8-8(FR) was formulated to account for the additional post yield capacity of reinforced concrete towers associated with inelastic behavior. Therefore, towers which meet the current criteria can be expected to resist a design level earthquake, or maximum credible earthquake where required, without collapse but with some structural damage.

The capacity and demand for internal shears and moments for the towers at Edward MacDowell Dam and Otter Brook Lake are presented in Tables 2 and 3, respectively. The demand, or required strength, for both towers was computed with the load combination shown in Equation 1 using a ductility factor of 1.0. Use of a higher ductility factor in Equation 1 was not warranted, since neither tower contains the minimum amount of flexural reinforcement, as required by the current criteria, to ensure a ductile failure. A review of the data contained in Tables 2 and 3 indicates that the tower capacity exceeds demand at Edward MacDowell Dam but that the moment capacity at the lower levels of the tower at Otter Brook Lake is well below the required strength.

Per the current criteria, the tower at Edward MacDowell Dam does not require remedial strengthening since its capacity exceeds the seismic demand. These criteria, however, are not satisfied at Otter Brook Lake, and a more refined ductility evaluation of the tower is required before making a determination as to whether remedial strengthening is required.

Table 2
Seismic Evaluation of Tower at Edward MacDowell Dam

Tower Section[1]	Shear Capacity kips	Shear Demand kips	Moment Capacity foot-kips	Moment Demand foot-kips
l	1,870	140	29,860	614
j	2,310	334	34,150	3,160
f	2,440	446	33,870	17,180
e	2,700	456	34,900	20,970

[1] See Figure 3 for location of sections

Table 3
Seismic Evaluation of Tower at Otter Brook Lake

Tower Section[1]	Shear Capacity kips	Shear Demand kips	Moment Capacity foot-kips	Moment Demand foot-kips
t	1,070	156	6,243	1,280
p	2,691	442	10,858	6,698
j	3,699	805	16,449	48,440
f	18,143	917	48,312	61,718
d	21,358	955	56,967	85,012

[1] See Figure 4 for location of sections.

Conclusions

Seismic evaluations of existing intake towers can be readily performed by the cantilever beam response spectrum method using the simplified two-mode added-mass procedure presented in current Corps guidance. Based on evaluations of the intake towers at Edward MacDowell Dam and Otter Brook Lake by this procedure, the following observations are offered. These observations are specific to towers in New England but appear to be generally applicable to all existing Corps towers which originally were designed by the traditional seismic coefficient method.

- The use of nonsite-specific earthquake data has the shortcoming of not accounting for the seismicity of the particular location. If an evaluation using nonsite-specific data indicates that a tower is deficient, it may be prudent to develop site-specific data before concluding that remedial work is required.

- In general, it appears that seismic forces computed by the analytical procedures outlined in the current guidance exceed the forces computed by the traditional seismic coefficient method. Therefore, in light of current criteria, the adequacy of all existing towers which were designed by the traditional method is in question.
- It appears that many existing towers do not contain sufficient flexural reinforcement to ensure a ductile mode of failure. An in-depth ductility evaluation of these towers will be required before any benefit of inelastic action can be included in a seismic assessment. Development of additional guidance on ductility is warranted.

References

American Concrete Institute. 1989. *Building Code Requirements for Reinforced Concrete*, (ACI 318-89), Detroit, MI.

Chopra, A.K. 1981. "Earthquake Forces for Design of Intake-Outlet Towers," manuscript submitted to the US Army Engineer Waterways Experiment Station, Vicksburg, MS.

International Conference of Building Officials. 1988. *Uniform Building Code*, Whittier, CA.

Krinitsky, E.L., and Dunbar, J.B. 1986. "Geological-Seismological Evaluation of Earthquake Hazards at Franklin Falls Damsite, New Hampshire," Technical Report GL-86-16, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Headquarters, Department of the Army. 1964. "Structural Design of Spillways and Outlet Works," Engineer Manual 1110-2-2400, Washington, DC.

Headquarters, Department of the Army. "Seismic Design and Evaluation of Intake Towers, " Engineer Technical Letter 1110-8-8 (FR), (in preparation) Washington, DC.

Vibro-Acoustic Study of an Aircraft Maintenance Dock

by

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Abstract

An analysis has been performed on the effects of high-level acoustic environments in an Aircraft Maintenance Dock (AMD). This analysis included detailed estimation of the maximum sound levels inside the facility, identification of the facility components which are potentially sensitive to high-level vibro-acoustic loads, and a summary of design considerations that are appropriate for this environment. The high noise levels represent the sum of direct sound pressures radiated by the internal noise sources impinging on the facility shell and the corresponding reverberant sound field inside the facility.

The maximum equivalent static pressures which would be expected to produce the same peak stress as the actual acoustic pressures inside the facility range from ±32 psf on the walls to ±27 psf on the roof, and an average of ±36 psf on a draft curtain. Dynamic response and fatigue effects are incorporated into these estimated acoustic loads. Dynamic (g) load factors for equipment components mounted on the facility structure vary widely depending on location and equipment weight and range from ±1.7 g's to ±17 g's. Additional details on vibro-acoustic design loads, including reaction loads on supporting structure due to acoustically induced vibration of wall and roof panels, vibration loads on the HVAC system, and random vibration test specifications for equipment mounted inside the AMD, have also been determined.

Several critical components of the AMD were analyzed, and recommendations are made to increase ductwork thickness and provide vibration isolation for ductwork, exhaust fans, pipe systems, light fixtures, and wind truss supports.

The type of standard steel construction to be employed for this facility, typical of high bay test facilities, is not normally exposed to the intermittent high-intensity vibro-acoustic loading anticipated from the planned test operations. However, if proper consideration is given to the vibro-acoustic loads specified herein for the design of the building shell and the mounting and/or qualifications of internal equipment, the planned utilization of the facility should not be significantly impaired.

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Introduction

Operations planned in the AMD will result in acoustic sound levels that may damage the structure or cause malfunction of other components in the facility, unless precautions are taken in the design of the secondary building structures and internal equipment mountings. An analysis of the impact of this acoustic environment in the facility has been completed and selected results are presented in this paper.

Facility Description

The total facility consists of two identical test bays (Figure 1), each composed of steel framing with braced perimeter columns supporting roof trusses that span East and West. Parallel to the trusses are steel joists to which the steel roof decking is attached. The walls are sheathed with 2-in.-thick insulated panels that are attached to horizontal steel girts by means of clips and bolts. The main facility doors consist of steel frames, with heavy steel panels fastened to the surfaces, and the cavities filled with insulation. A draft curtain, consisting of a heavy roof deck supported vertically by steel frame members, creates a cavity for capturing exhaust fumes from the aircraft auxiliary power units (APU). All of the steel members are joined together by welded and bolted connections.

Acoustic Analysis

The vibro-acoustic environmental design analyses in this study are based on a maximum operating condition of 6,820 RPM for four primary noise sources and for two auxiliary power noise sources operating at maintenance power at the same time. The 6,820 RPM condition for the primary sources was the power condition specified for design purposes of this study.

Development of analysis approach

For the various operating conditions of the acoustic sources involved in the AMD, the acoustic pressures on the structure are a composite sum of the direct radiation from each of the sound sources, accounting for the effect of the first reflection (i.e., pressure doubling) at the interior surfaces and the reverberant sound field caused by the acoustic energy remaining after the first reflection of the direct sound. The direct sound field was determined from the noise contours provided by the data report on the primary noise source.

The reverberant sound levels depend on the total sound power generated by the sources and on the acoustic absorption at the interim surfaces of the AMD. When the source position is moved forward by the vehicle taxiing out, the maximum levels in the middle one-third of the roof structure and in the draft curtain will tend to be controlled by the direct field. Therefore, increasing the roof absorption (or adding acoustic absorption on any internal surfaces, including the walls) will not have a major effect on maximum (design) sound levels in this middle portion of the roof.

Acoustic design environment

The acoustic design environment for the AMD corresponds to the upper envelope in Figure 2. The following equation converts this environment to pressure (P):

$$P = 4.177 \times 10^{-7} \times 10^{\left(\frac{L_b}{20}\right)}, \text{ psf} \qquad (1)$$

where

P = pressure in pounds per square foot

L_b = sound pressure level in dB

Vibro-Acoustic Response

The vibro-acoustic response will be considered in four forms: (1) acoustically equivalent static pressure loads on secondary wall, roof and door panels of the AMD shell, ductwork panels, and lightweight equipment covers; (2) vibration load factors for design of mounting structure for internal mechanical, electrical and hydraulic equipment; (3) vibration or acoustic test environment specifications that may be required in procurement specifications

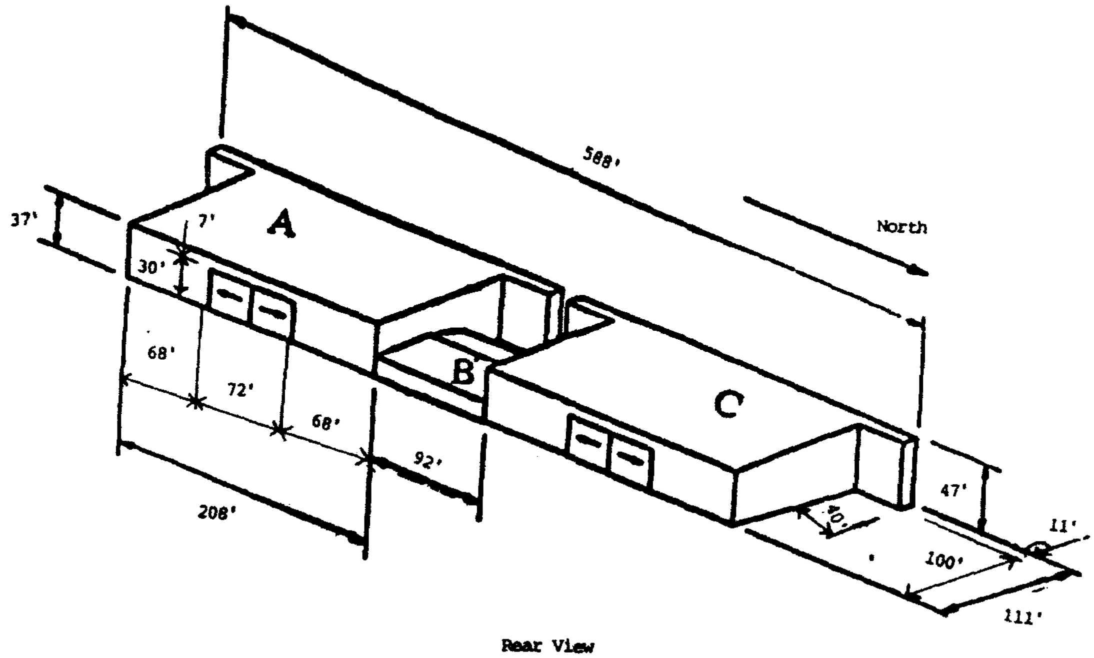

Figure 1. Aircraft maintenance dock

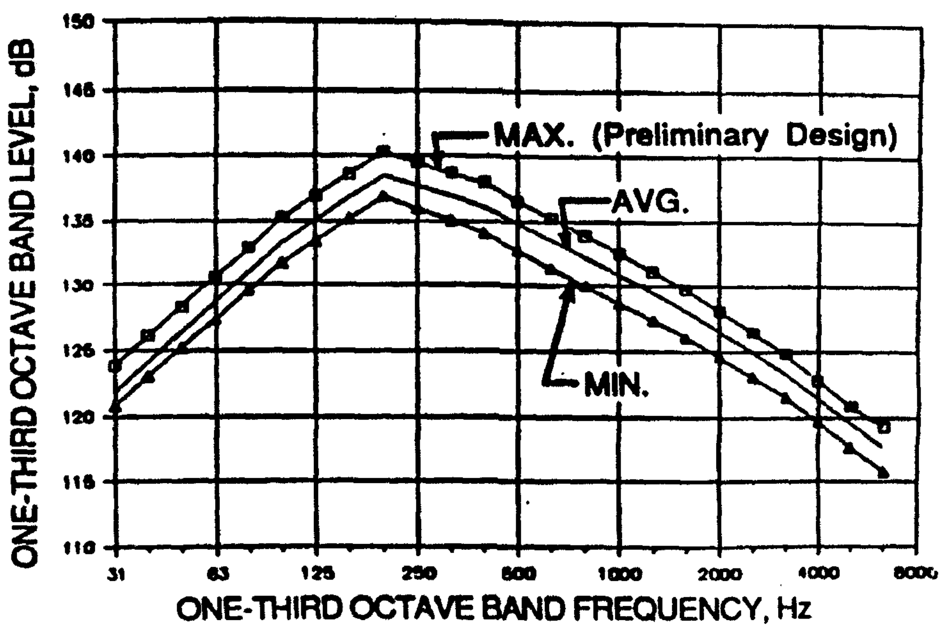

Figure 2. Acoustic design environment in one-third octave band sound pressure levels

for equipment that cannot be preselected to be assuredly capable of withstanding the vibro-acoustic environment resulting from normal operations in the AMD; and (4) equipment and secondary structure vibration isolation.

The secondary skin structure elements of the AMD and lightweight enclosures of AMD equipment have two common characteristics that are the primary cause of the acoustically-induced vibration response: their large surface area and low surface weight.

Equivalent static pressure loads

The spatial average root mean squared (RMS) acceleration response ($\tilde{a}_{m,n}$) of a simply supported plate vibration in the m,n mode, excited by a broad band random acoustic excitation is given in the following equation by Wilcoski and Sutherland (unpublished):

$$\tilde{a}\,(f_{m,n}) = \sqrt{\frac{\pi}{2}\,\frac{P_b^2\,(f_{m,n})\,J_{m,n}^2\,(f_{m,n})\,Q}{(0.2316)w^2}} \qquad (2)$$

where

$P_b^2\,(f_{m,n})$ = mean square pressure in a one-third octave band centered on the resonance frequency, $f_{m,n}$, of the plate

$J_{m,n}^2\,(f_{m,n})$ = joint acceptance for a simply supported plate vibrating in the m,nth resonance frequency mode, under a diffuse sound field (Sutherland 1968)

w = plate surface density

The spatial average RMS velocity $\tilde{v}\,(f_{m,n})$ response for each mode to this excitation is given by:

$$\tilde{v}\,(f_{m,n}) = \tilde{a}\,(f_{m,n}) \times \frac{g}{2\pi f_{m,n}} \qquad (3)$$

The spatial maximum RMS velocity ($\tilde{\dot{X}}_{max}$) *at the center of a simply supported* plate will be four times the spatial average RMS response ($\tilde{v}\,(f_{m,n})$) over the entire surface of the plate. The maximum RMS stress $\tilde{\sigma}_{m,n}$ (max) in each mode is given in Equation D17 by Wilcoski and Sutherland (unpublished):

$$\tilde{\sigma}_{m,n}\,(\max) = K_s\sqrt{3}\,E\,\frac{\tilde{\dot{X}}_{max}}{C_L} = K_s\sqrt{3}\,\rho C_L \tilde{\dot{X}}_{max} \qquad (4)$$

where

K_s = shape constant that depends on panel edge constraints and aspect ratio

E = modulus of elasticity, psi

C_L = longitudinal speed of sound in the plate material, in/sec = $(E/\rho)^{1/2}$

ρ = mass density of the plate material, lb sec^2/in.^{4}

The overall maximum RMS stress $\tilde{\sigma}_{max}$ is given by:

$$\tilde{\sigma}_{max} = \sqrt{\sum_{m}^{M}\sum_{n}^{N}\tilde{\sigma}_{m,n}^2\,(\max)} \qquad (5)$$

The equivalent static pressure P_s which produces the same peak stress, taking into account the random peaks of the excitation and fatigue effects is given in the following equation by Wilcoski and Sutherland (unpublished):

$$P_s = \frac{F_p}{F_f K}\left(\frac{t}{a}\right)^2 \tilde{\sigma}_{max} \qquad (6)$$

where

F_p = ratio of peak over RMS response, at 10% exceedance $F_p = 2.15$

F_f = fatigue reduction factor, for 1,200,000 panel cycles in 4,000 operations of the facility, over its Design Life, $F_f = 0.75$

K = function of edge constraints and aspect ratio (differs from K_s)

t = plate thickness

a = short side width of the plate

Table 1 summarizes the results of these calculations for several components in the AMD.

Table 1
Equivalent Static Pressure for Design of Lightweight Structural Elements on AMD Subjected to Acoustic Design Environments

	Estimated Maximum		
Element	**Peak Deflection In.**	**RMS Velocity In./sec**	**Static Pressure P_s, psf**
Wall Panels	0.38	19	±32
Roof Panels	0.36	5.7	±27
Draft Curtain	0.22	15	±34

Vibration loads for design of AMD equipment and mounting structure

Vibration loads for AMD equipment and mounting structure are defined in terms of (a) vibration load factors (LF), i.e., g-loads, to be applied to the design of any structure-mounted AMD equipment and its mounting attachment, and (b) reaction loads on secondary framing structure due to vibration of wall or roof panels. In all cases, these loads are bidirectional, that is, they are applied in each direction along any one axis.

The vibration load factors deal with the overall panel vibration and are therefore based on spatial average rather than the spatial maximum response in the center of a panel. The acceleration values are further modified because the surface weight parameter used in the response equations includes the added weight of the support structure distributed over the surface of the acoustically loaded structure in question. Thus, the space average acceleration levels for the wall structure are based on the total average weight of the entire wall system, panels plus girts and columns, distributed over the surface. Mass loading effects of heavy equipment mounted on the building frame serve to further reduce the acceleration levels of the structure. This reduction is defined by the ratio $W_m/(W_c + W_m)$, where W_e is the total weight of equipment mounted on the structure and W_m is the total effective weight of the basic structure on which the equipment is mounted. Values for W_m are indicated in Table 2 for primary and secondary structural members in the AMD. The effective weight W_m of the mounting structure represents the dynamically effective weight of a vibrating beam, which is 50 percent of the true weight. A few examples of the results of these calculations are summarized in Part (a) of Table 2. It is recommended that equipment be attached to or near primary structural members to avoid the high vibration levels, indicated in Table 2 for the center of wall, roof, and draft curtain panels.

Examples of the vibration reaction loads are summarized in Part (b) of Table 2. Reaction loads in the vertical direction are defined for the up and down direction taking into account the inherent -1 g download due to gravity. Load factors and vibration reaction loads in Table 2 (a) and (b) are in the direction of the vibrating surface; in the perpendicular directions these responses become 50 percent of the basic value.

Table 2
Loads for Design of Supporting Structures for AMD Secondary Structure and Equipment Subjected to Acoustic Design Environment

(a) Vibration Load Factors

Mounting Location	W_m Effective Weight of Mounting Structures, lb	Baseline Vibration Design Load Factor[1]
Wall Panel (2.5' x 7.5')	26 lb/panel	17
On Girt, Back Wall	715	8
On Columns, Back Wall	5035	5.4
Roof Panel/Bar Joist Section	520	5.2
Roof Truss	5017	3.1
Draft Curtain - Panel	300	21
Bottom Channel	1260	5.0
Roof Girders	5300	2.8

(b) Vibration Reaction Loads

Joint Vertical Surfaces	Perpendicular to Surface	Co-Planar with Surface		
		Up	Down	Horizontal
Wall Panel/Girt	±610[2]	±160	-440[2]	±303[2]
Girt/Column	±1235[2]	0	-1300	±176
Horizontal Surfaces		**Up**	**Down**	**Horizontal**
Roof Deck/Bar Joist		+25 lb/ft^2	-610 lb/ft^2	±160 lb/ft^2

[1] Load factors in direction normal to plane of surface (i.e., wall, roof, etc.). For in-plane vibration loads, multiply factors by 0.5.
[2] Reaction load increased for stress concentration by factor of 3.5.

Ductwork loading, motion, and recommendations

The dynamic response of all ductwork panels was analyzed using the methods described above under the acoustic load conditions defined in Figure 2. This analysis included all modes of vibration up to 1,000 hz. Table 3 summarizes these results for a few ducts, with the peak displacement (X_{pk}), load factors (LF), peak stress (σ_{max}), equivalent static pressure (P_s), and support reactions. Various duct panel thicknesses were evaluated

Table 3
Ductwork Loading and Motion

Duct Size, in.	Panel Size, in. W	Panel Size, in. L	Ga	F, Hz	W/120, in.	X_{pk}, in.	LF, g	σ_{max}, psi	P_s, psf	Reactions, lb/ft Long Edge	Reactions, lb/ft Duct Support
Horizontal											
36 x 36	36	133	14	5.9	0.300	0.35	7.9	5100	4.0	110	89
			16[1]	4.7		0.45	9.5	6300	3.0	114	85
			18	3.8		0.57	9.5	7700	3.0	101	68
Vertical											
14 x 8	8	90	16	89.5	0.067	0.07	14.7	10500	111.9	13	90
			18[1]	71.6		0.11	19.0	13700	93.5	14	93
			20	53.7		0.19	29.0	19300	76.5	17	106
72 x 12	72	90	11	3.6	0.600	0.63	2.4	2900	2.9	91	29
			14[1]	2.2		1.68	2.4	9600	3.8	12	18
			16	1.8		2.50	3.7	5700	1.4	70	22

[1] Gauge (ga) recommended by USACERL.
[2] Dynamic Magnification factor (Q) is conservatively set to 30.

and the gages (Ga) recommended by US Army Construction Engineering Research Laboratory are indicated by a superscript 1.

Acoustic and vibration environmental test specifications

Acoustically or vibration sensitive equipment may need to be experimentally qualified. The recommended acoustic test is conducted by mounting equipment inside a reverberant acoustic chamber and testing the equipment according to the environment defined by the upper envelope in Figure 2 for 16.7 hours. The vibro-acoustic loading is based on 4,000 operations of the noise sources for 7 to 15 seconds per operation, over the life of the facility. This equates to 8.3 to 16.7 total hours of operation, and the 16.7-hour test will provide an adequate qualification test.

The vibration tests should be conducted by attaching the equipment in its typical mounting configuration to a vibration test platform. The equipment should then be tested for 10 hours to the appropriate acceleration power spectral density (ASPD) envelope. Examples of such envelopes, shown in Figure 3, are for equipment mounted at various locations on the AMD wall. The ASPD levels in Figure 3 are in the direction normal to the wall surface; in the perpendicular directions these responses become 25 percent of the basic value, because the acceleration levels are reduced to 50 percent and the ASPD envelopes are in units of g^2/Hz.

Building component vibration isolation

Several equipment and secondary structural components should be isolated from the high vibration levels, both to protect them and to reduce the load on building and mounting substructures. The vibration isolation recommendations are based on the knowledge that the support points will vibrate at well understood minimum frequencies, based on the lowest natural frequency of the supporting system that is being driven by acoustical loading. For example, most of the items being isolated are supported at either the roof bar joists or wall girts, where these members will have a lowest mode of vibration at those frequencies defined by the roof panel/bar joist or wall panel/wall girt dynamic response.

The dynamic responses of the isolated components are modeled as simple spring/mass single degree of freedom (SDOF) systems driven by the support motion described earlier. The vibration at various support locations was defined as an envelope of acceleration levels at each of the panel natural frequencies. For each component being supported, maximum (W_{max}) and minimum (W_{min}) static loads per isolator were determined. The transmissibility (T) is the ratio of dynamic response to dynamic input. A reasonable goal is to isolate components with springs soft enough to limit the transmissibility through the spring for the lowest support motion (driving) frequency to 0.2. The transmissibility at a particular driving frequency (f_d) of an SDOF oscillator with a natural frequency (f_n) and damping (ξ) is given by the following equation (Thomson 1981):

$$T = \frac{\sqrt{1 + \left(2\xi \frac{f_d}{f_n}\right)^2}}{\sqrt{\left[1 - \left(\frac{f_d}{f_n}\right)^2\right]^2 + \left(2\xi \frac{f_d}{f_n}\right)^2}} \qquad (7)$$

The transmissibility is set to 0.2, for the lowest support motion frequency, and the equation is solved for the resulting maximum natural frequency (f_n) of the isolated system. This value, along with the minimum load of the SDOF system (W_{min}) is used to calculate the maximum acceptable stiffness (k_{max}) of the isolator that will limit the transmissibility to 0.2. The maximum stiffness is given by the following equation:

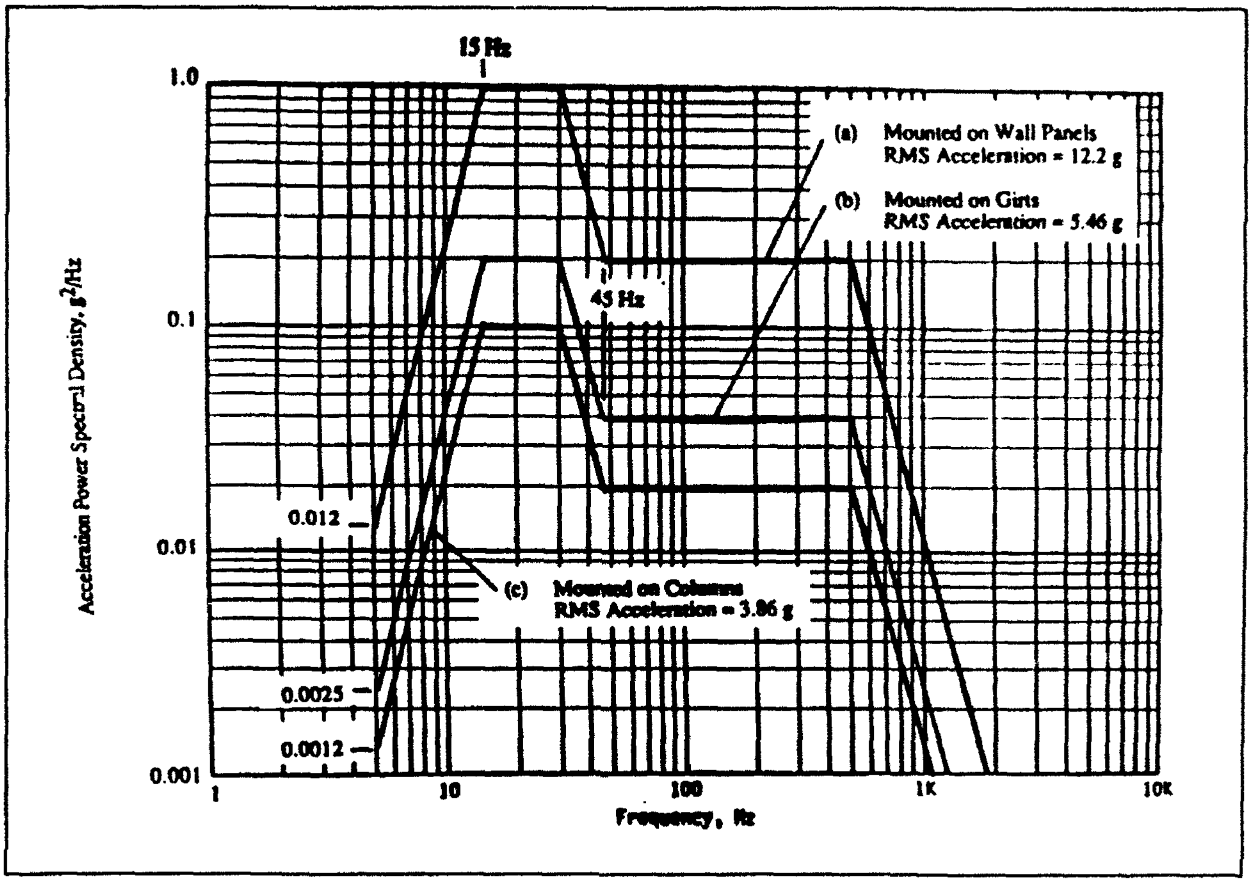

Figure 3. Vibration test acceleration power spectral density (ASPD) envelopes

$$k_{max} = 2\pi f_n \left(\frac{W_{min}}{g} \right) \tag{8}$$

From this maximum stiffness, a commercially available isolator is selected. The stiffncss of each isolator is used to calculate the actual natural frequency of the isolated system. The acceleration of the isolated system is then calculated from the peak acceleration for each mode of vibration at the support location. For each mode the actual transmissibility is calculated using Equation 7. If k_{max} is not exceeded, the transmissibility for the fundamental mode of vibration will be less than 0.2. For all other modes the transmissibility is less than 0.1, but we know that some excitation will pass through even at the higher modes, so we conservatively set the minimum transmissibility to 0.1 for all but the first mode. The effective acceleration (a_{eff}) is then calculated as the square root of the sum of the squares of the peak acceleration for all modes ($a(f_i)$) times their transmissibility factors ($T(f_i)$), all times a reduction factor for mass loading of the support structure. This calculation is expressed by the following equation:

$$a_{eff} = \sqrt{\sum [(a_{(f_i)}) \times (T_{(f_i)})]^2} \times \left(\frac{W_m}{W_e + W_m} \right) \tag{9}$$

Table 4 summarizes the results of a spreadsheet program used to calculate the dynamic response and effective dynamic vertical (V_{max}) and horizontal (H_{max}) loads for a few of these isolated SDOF systems. Figure 4

Table 4
Recommended Vibration Isolation

Item	W_{min}, lb	W_{max}, lb	T	f_d, Hz	f_n, Hz	K_{max}, lb/in.	ξ	a_{eff}, g	V_{max}, lb	H_{max}, lb	Iso Type
(a) Supported at the Roof Bar Joists											
a[1]	482	1009	0.2	9.85	4.02	797	0.005	0.395	1408	399	E
b	110	320	0.24	9.85	4.36	214	0.005	0.429	457	137	G
c	200	238	0.23	9.85	4.26	371	0.005	0.293	307	70	P
(b) Supported by the Wall Girts											
d	170	240	0.2	11	4.49	351	0.005	0.813	435	195	N
(c) Supported by the Roof Truss											
e	50	60	0.26	9.85	4.47	102	0.005	0.300	78	18	A

[1] a – Horizontal wind truss support with single isolator to three bar joists.
b – Horizontal ductwork with two isolators to two bar joists.
c – Exhaust fans 1, 2, 11, and 12 and attached ductwork with four isolators.
d – 6" horizontal oscillating monitor fire protection pipe.
e – 55-lb HID light fixture with single isolator.

illustrates a typical vibration isolation support for a High-Intensity-Discharge (HID) light fixture.

Summary

The acoustic environment from aircraft operation at 6,820 RPM would be expected to cause some secondary structural failures or equipment malfunction if the design loads developed in this study are not accounted for in the design. Maximum static pressure design loads equivalent to the acoustic environment will be as high as ±32 psf on the back wall. Dynamic load factors due to acoustically induced vibration will vary depending on mounting location and equipment mass. For equipment not mounted on wall, roof, or draft curtain panels, vibration load factors are estimated to be no greater than 8 g. Several strengthening measures, such as increasing ductwork panel thickness, should be taken.

Several equipment and secondary structural components should be isolated from the high vibration levels, both to protect the items being isolated and to reduce the loads on the building and mounting substructures.

References

Sutherland, L. C. 1968 (Mar). "Sonic and Vibration Environments for Ground Facilities - A Design Manual," Wyle Laboratories Report WR 68-2, El Segundo, CA.

Thomson, W. T. 1981. *Theory of Vibration with Applications*, 2nd ed., p 65, Prentice-Hall, Englewood Cliff, NJ.

Wilcoski, J., Sutherland, L. C. "Vibro-Acoustic Analysis of a Aircraft Maintenance Dock," unpublished, Equations D11, D17, and D35, US Army Construction Engineering Research Laboratory, Champaign, IL.

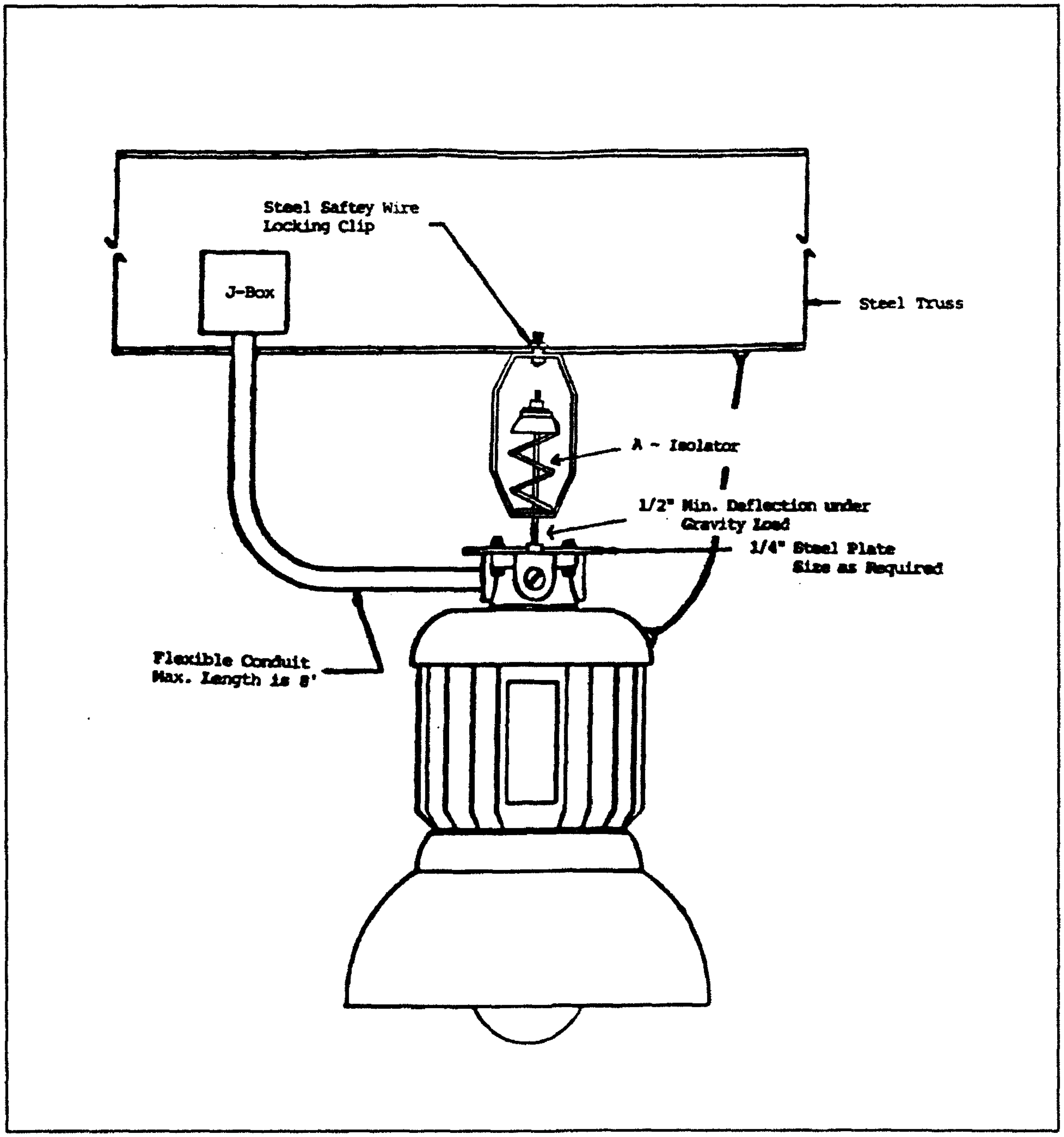

Figure 4. HID light fixture support detail

Nondestructive Evaluation of Masonry

by
Robin C. Westerfield, PE[1]

Abstract

A large number of older buildings were constructed of unreinforced clay brick masonry, while more recent structures often use brick veneer or concrete masonry as an exterior wall finish. In many cases, these buildings must be analyzed for structural adequacy due to change of use, historical preservation, or upgrading to meet seismic requirements. Recently constructed buildings may need a structural evaluation when the quality of workmanship or materials is suspect. Traditional evaluation of existing masonry has involved destructive testing of specimens removed from the structure. However, in addition to the possible aesthetic and structural problems associated with removing test prisms from a structure, there are no standards available to evaluate the specimens. Nondestructive evaluation (NDE) methods can potentially aid in rapid evaluation of large areas of a structure while eliminating or minimizing damage caused by removal of samples for destructive tests. This paper discusses some of the possibilities for masonry evaluation and the testing procedures using NDE methods.

Introduction

Evaluation of masonry buildings for structural integrity, load capacity, and dynamic response includes consideration of building geometry, site conditions, loading, connections between walls, and connections between walls and floors or roofs, openings, reinforcement and masonry material properties and condition. Obviously, there are many other considerations than masonry quality alone when assessing the structural adequacy of a building. However, as a high percentage of these structures use masonry to resist both vertical and horizontal loadings, evaluation is essential in determining whether a costly new structural system must be added or even if renovation is practical. Since the engineer will often be working without plans or calculations from the original design, particularly in older buildings, nondestructive evaluation (NDE) techniques may convince the designer that the masonry portions of the structure can indeed be used without costly alterations.

Application of NDE techniques to masonry has traditionally involved adapting methods which have proved successful in the evaluation of concrete and rock. Masonry NDE techniques can be divided into those intended to measure material condition, and those used to measure mechanical properties. The latter can be further divided into two general categories: (1) "indirect" tests in which masonry mechanical properties are estimated by correlations to nondestructive measurements, and (2) "direct" physical measurements of mechanical properties. The term "properties"

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used herein refers to the mechanical properties of masonry such as deformation properties, compressive strength, tensile strength, mortar-to-unit shear strength, modulus of rupture (out of plane), and structural durability. The term "material condition" refers to the presence of cracks, voids, material deterioration, other flaws in masonry, and state of stress.

Flaws that have been observed in masonry include:

- Delaminated collar joints in multi-wythe masonry. This flaw destroys the composite action between wythes and may cause a wall to be vulnerable to out-of-plane loadings.
- Delaminated bed joints. These may reduce resistance to horizontal loadings and provide paths for penetration of water through the wall.
- Diagonal tensile or "shear" cracking. Earthquake loading or uneven foundation movement may cause this problem.
- Isolated zones of "failed" masonry which may be caused by overstress or environmental degradation such as would be produced by sulfate attack or by freeze-thaw cycles in a constantly wet portion of a wall.

The following sections review past and ongoing research on a number of techniques which offer the greatest potential for masonry evaluation. Of these methods, only the flatjack methods and the in-place shear test can be considered "direct" methods. The others require some estimation on the part of the engineer.

NDE Test Methods

Schmidt Hammer

This test is the quickest and least expensive method for nondestructive evaluation of solid clay (brick) masonry. However, its utility is limited and it is recommended only for determination of the uniformity of properties over a large area of a structure. It evaluates only the local point and layer of masonry to which it is applied, and is unreliable for detection of flaws or for investigation of inaccessible masonry wythes.

The Schmidt Rebound Hammer provides a measure of relative material surface hardness. It has been used extensively in the testing of concrete and rock. The hammer consists of a spring loaded plunger which, when released, strikes a surface and causes a mass within the hammer to rebound. The magnitude of the rebound is indicated on a scale (the rebound number), and gives an indication of surface hardness which can be correlated to the strength or condition of the material. While a relationship may exist between rebound number and masonry compressive strength it is not recommended that the Schmidt Hammer be used for direct prediction of compressive strength but only for evaluation of material uniformity.

Flatjack methods

These tests provide perhaps the most powerful tools for the nondestructive evaluation of masonry. A flatjack is a thin steel bladder that is pressurized with oil to apply a uniform stress over the plane area of the flatjack. In masonry structures, flatjacks are inserted in slots cut in mortar bed joints. The flatjacks may be made in different shapes and sizes—flatjacks with curved edges are designed to fit in a slot cut by a circular masonry saw, and rectangular jacks are used where mortar must be removed by hand or with a drill. Semicircular jacks are suitable for in-situ stress measurement but are not suitable for deformation measurements in the two-flatjack test. For deformation tests, rectangular flatjacks with lengths equal to at least that of two masonry units should be used.

ASTM standards are currently being finalized for the application of flatjack testing to the evaluation of unreinforced solid clay unit (brick) masonry. Under the proper conditions, flatjacks can provide information on the

in-situ state of stress at virtually any point in a masonry structure, a measure of the deformability of the masonry materials, and in some cases a direct measure of masonry compressive strength. The flatjack tests are not strictly nondestructive, since they require the removal of some portion of a mortar joint. However, this damage is easily repaired by simply repointing mortar into the slot, leaving no visible trace of the test. The two main types of flatjack tests as described in the following paragraphs are the in-situ stress or single-flatjack test, and the in-situ deformability or two-flatjack test.

- **In-situ stress test**. When the mortar is removed from a horizontal joint, the release of the stress across the joint causes the slot to close by a small amount. The magnitude of this deformation is measured using a removable dial gauge between two or more points located symmetrically on either side of the slot. A flat jack is then inserted in the slot and pressurized until the original position of the measuring points is restored. The pressure in the flatjack, when modified by two constants to account for the flatjack stiffness and the area of the slot, is assumed equivalent to the original vertical compressive stress in the masonry (see Figure 1). Past results show that the in-situ stress test is able to estimate the actual state of masonry compressive stress within 10 to 15 percent.

- **In-situ deformability**. The deformation properties of masonry may be evaluated by inserting two parallel flatjacks, one directly above the other separated by several courses of masonry, and pressurizing them equally, thus imposing a compressive load on the intervening masonry. The deformations of the masonry between the flatjacks are then measured for several increments of load, and used to calculate the masonry deformability modulus (see Figure 2). If some damage to the masonry is acceptabl-, the masonry may be loaded to failure to determine the maximum strength. This technique is useful when an estimate of material deformability or strength is needed for stress analysis or deflection calculations. The in-situ deformability test provides a reasonably accurate measure of masonry compressive modulus, typically overestimating the masonry stiffness by about 10 percent.

- **In-situ shear**. The in-place shear test, often called the "push test", measures the in-situ joint shear resistance between masonry units and mortar joints. It requires the removal of a single masonry unit and a head joint on opposite sides of a test unit. The test unit is then loaded horizontally by a hydraulic jack, and the horizontal force required to cause first movement of the test unit is recorded. The test may be considered nondestructive, as the removed unit and mortar joints may be replaced and restored to their former appearance.

The in-place shear test is the best means currently available for measuring in-situ bed joint shear strength in existing masonry walls. A number of assumptions must be made, however, if reliable results are to be obtained from the test—these include the definition of joint failure, the effect of normal load on the measured shear stress, the magnitude of the normal load on the tested joint, the contribution of the collar joint, the variability of the masonry due to workmanship in the original construction, and correlation to full-scale wall behavior. Judgment will be required by the engineer to convert the measured stresses into the actual capability of the wall to resist shear.

A modified technique for conducting the in-place shear test has been developed which addresses many of these assumptions and appears to give reliable results. In the modified test, the vertical stress in the wall at the test unit is measured directly using the single flatjack test, and the normal stress on the test unit during the test is controlled by flatjacks above

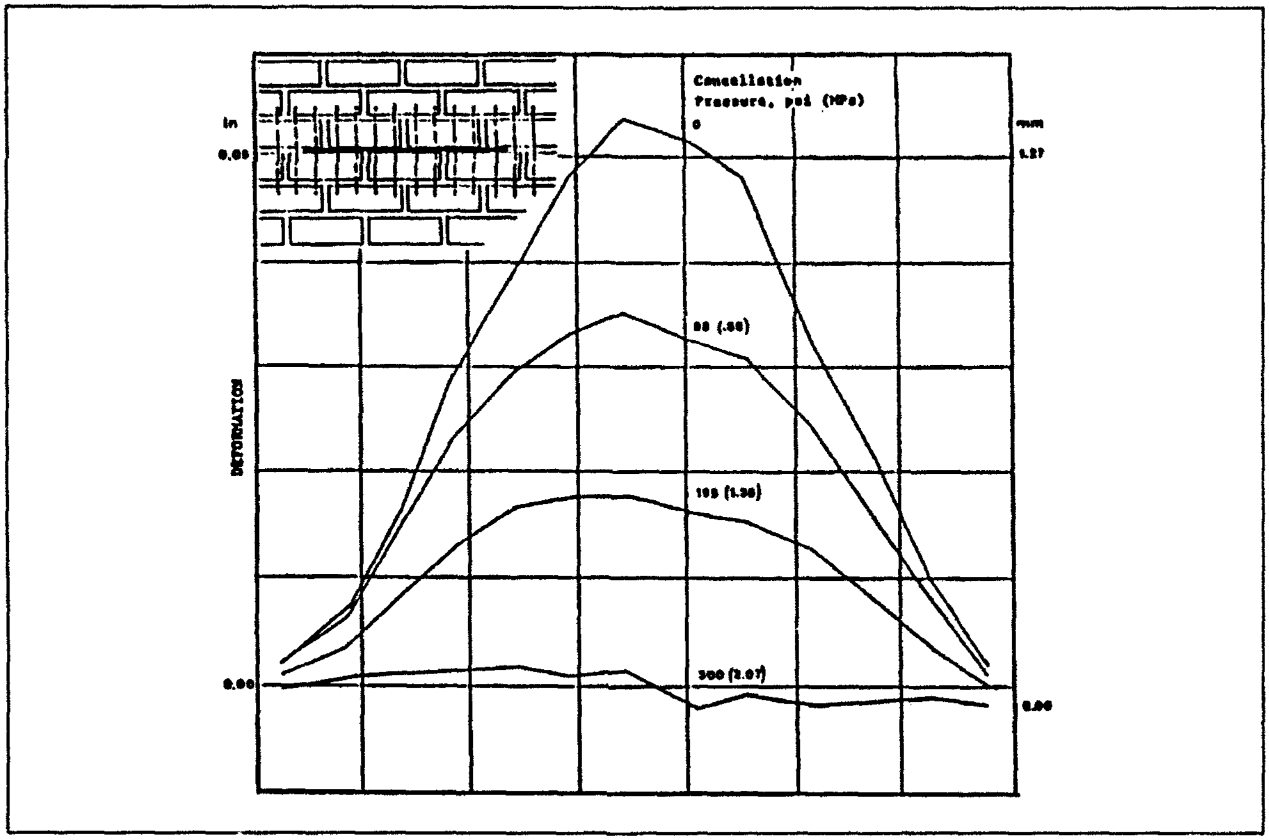

Figure 1. Masonry deformations around flatjack slot during in-situ stress test

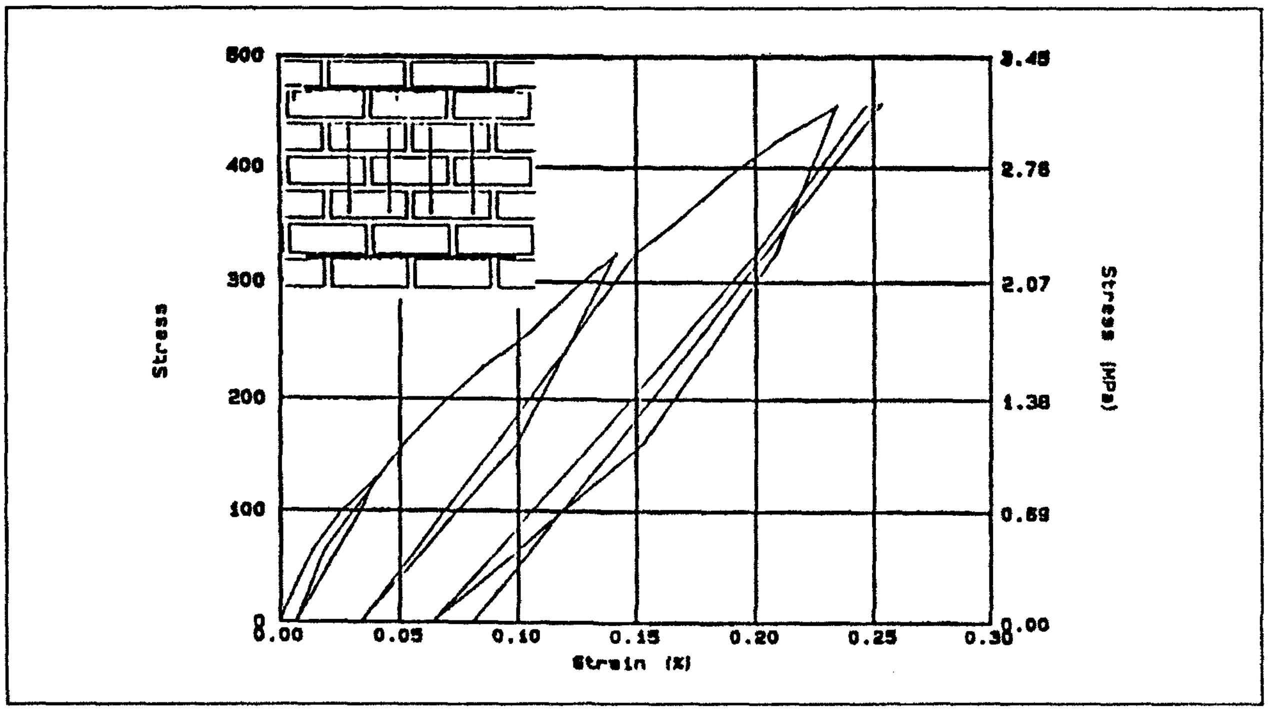

Figure 2. Stress-strain curve obtained during in-situ deformat˙˙ity test

and below the test unit (see Figure 3). The test is then conducted on the same joint for several levels of normal stress, so the friction angle is measured directly rather than assumed (see Figure 4). Strain gages are used to monitor the movement of the unit continuously during the test, thus eliminating ambiguity concerning the definition of failure. The collar joint shear strength may be estimated by a shear test, leaving only the consistency of workmanship as a potential source of error.

Because the in-place shear test measures the bedjoint shear strength directly with a minimum of damage to the structure, it is an essential part of any building evaluation where lateral loads are expected to influence the building design. In some seismic regions the existing test is already required for some retrofit designs. The modified test should be conducted as an extension of a normal series of flatjack tests. The single flatjack test reveals the in-situ state of normal stress at the test joint, providing essential data for determining the expected joint shear strength in the area of the test. The two-flatjack test then provides half of the required test setup for the modified in-place shear test. At the completion of the test, the engineer will know the relationship between the expected joint shear strength and normal stress, and also the measured normal stress at the test location. If a simpler method such as the Schmidt Hammer test can establish similarity of materials throughout the structure, the number of required in-place shear tests can be reduced from a fixed number such as so many tests per square foot. The single-flatjack test could be used predominately to determine the variation of normal stresses throughout the structure.

- **Ultrasonic pulse velocity**. The ultrasonic pulse velocity (UPV) method uses two transducers (transmitter and receiver) and a power unit with a digital transit time display to pass a high frequency (50,000 Hz) stress wave through masonry. It is most useful in locating relatively small flaws in otherwise uniform masonry construction. While it may be possible to obtain an estimate of compressive strength with this method, other testing is recommended in order to interpret the data properly. The flatjack test could be used for determining the state of stress and deformability in a wall to provide a baseline calibration for UPV testing, for instance. The use of this method for concrete evaluation has shown that many factors can affect the pulse transmission time such as aggregate type and size, moisture content, and the presence of reinforcement. Generally, those factors which can affect compressive strength may also affect ultrasonic pulse velocity, though not necessarily in direct proportion. Strength predictions can only be justified if a calibration of pulse velocity with masonry strength is made for each structure under consideration, and then only if the conditions of testing can be carefully controlled. Because of this limitation, the pulse velocity method is generally used only to measure material uniformity over a large area of a structure. Lower frequency sonic testing (1 to 5 kHz), or mechanical pulse testing, seems to hold more promise in the development of nondestructive evaluation techniques for masonry structures.

- **Mechanical pulse velocity**. This method involves input of a stress wave into a masonry wall by means of a hammer blow and recording the resulting vibrations with an accelerometer. Due to its low frequency, high amplitude, and long wavelength signal, this technique is better suited to the evaluation of masonry than the ultrasonic technique. The equipment required for the test includes a 3-lb instrumented hammer and an accelerometer as well as equipment to record the data. A typical setup is shown in Figure 5. Since there is no digital read-out of travel time with this equipment, the signal must be recorded or displayed on an external device. A digital transient recorder can be used to record both the hammer input

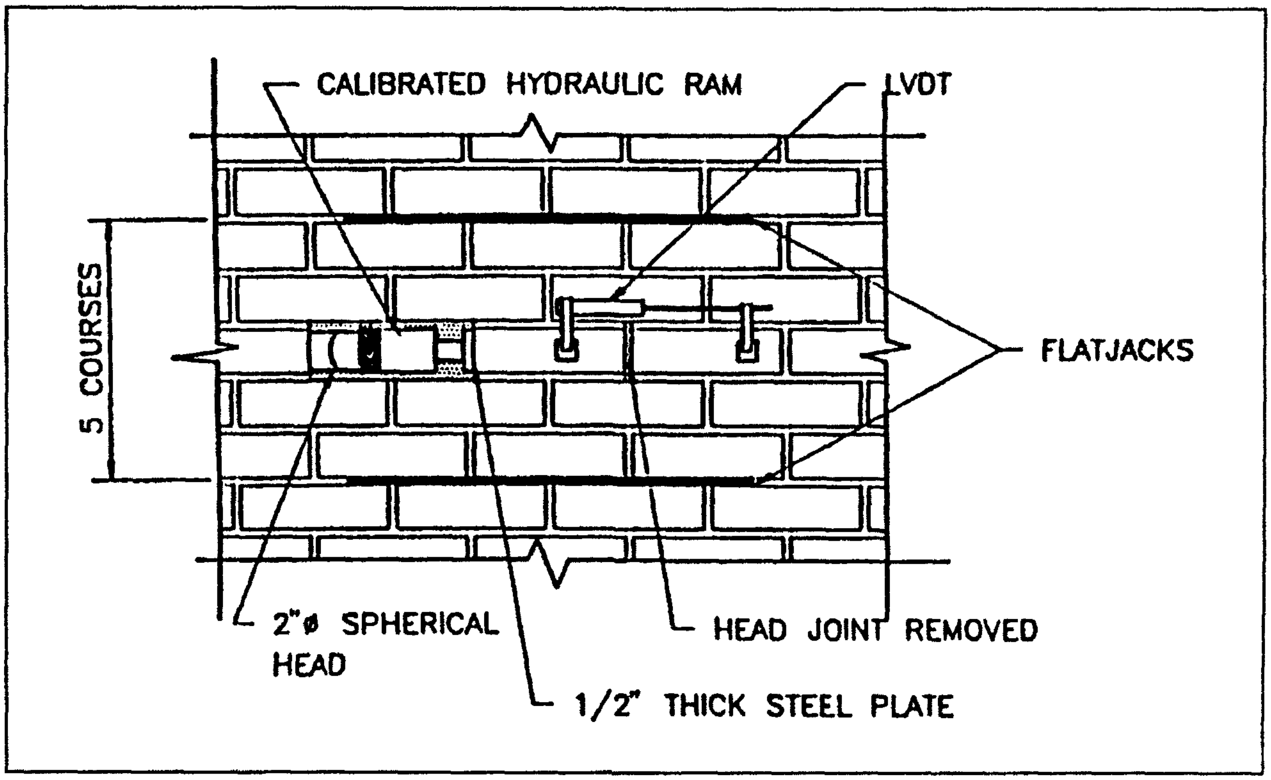

Figure 3. Setup for modified in-place shear test

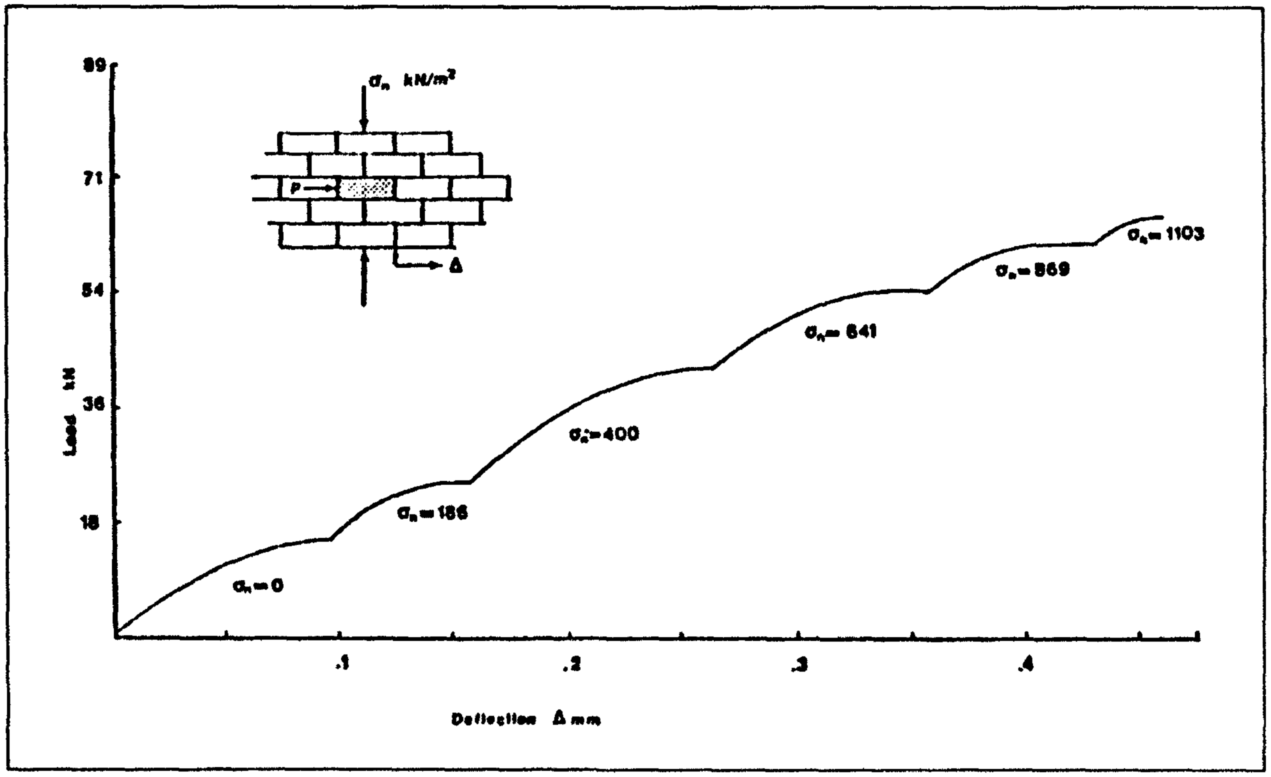

Figure 4. Results from an in-place shear test using modified procedure, showing relationship between measured bedjoint shear stress and horizontal displacement

signal and the accelerometer output signals. Alternatively, an oscilloscope may be used to measure travel time.

The mechanical pulse technique is best suited for locating flaws and discontinuities such as missing mortar joints and large cracks and establishing relative quality of masonry from one location to another. Figure 6 shows the effect of a delaminated bed joint on mechanical pulse velocity. The high energy and long wavelength of the input pulse (as compared with the ultrasonic pulse method) are not as rapidly attenuated by the boundaries between units and mortar that are integral parts of masonry construction. Because of this, the mechanical pulse will travel farther through most masonry materials than the ultrasonic pulse and can detect the larger flaws that are of interest in a structural evaluation. It is recommended that mechanical pulse tests be conducted in conjunction with flatjack tests, so that the affect of varying vertical stresses and material deformability on the mechanical pulse measurements can be assessed.

- **Location of reinforcement and ties.** The use of magnetic and resistance methods allow quick inspection of masonry construction for the presence of steel reinforcement or ties. These techniques may be useful for quality control as a means of verifying compliance with construction plans, and provide reasonable results when expected reinforcing bar sizes and locations are known. More difficult is the case of renovation projects, when it is necessary to not only locate the reinforcement but also estimate the size and depth of the bar. Since a weak signal can indicate either a small bar close to the surface or a larger bar located farther from the probe, it may be necessary to expose the reinforcement at trial locations to verify assumptions regarding size and location.

  Commercially available meters use an electromagnetic field generated around a hand-held probe to indicate the presence of steel in the masonry. A voltage change occurs when the field is interrupted by a ferrous material such as steel reinforcement, with the magnitude of the voltage change being proportional to the amount of steel and the distance from the steel to the probe. Since the meters will locate all steel present, not just the reinforcing bars, some care needs to be taken not to identify metal ties, nails, electrical conduit, etc., as reinforcing steel.

- **Nuclear methods**. Although not related directly to structural properties of materials, the Neutron-Gamma technique shows great promise for certain aspects of masonry evaluation. The technique measures element concentrations in masonry walls and, thus, gives information about moisture content, presence of salts, and elemental composition of the masonry materials. The technique has been shown to be complementary to structural evaluation techniques by aiding the interpretation of results from tests such as the mechanical pulse technique.

Application of Combined NDE Techniques

The NDE of masonry for mechanical properties and condition is quite difficult, because the heterogeneous and highly variable nature of the material hinders the simple analysis and interpretation of test results. At the current level of development, the best application of NDE to masonry would make use of a number of complimentary techniques. For example, rapid methods such as the Schmidt Hammer might be used to assess the condition of the entire structure, and pulse velocity methods would then be used to map the variation in material condition in critical areas of the structure. Direct mechanical measurements of material deformability and joint shear strength might then be made in locations defined during the ultrasonic or sonic pulse velocity mapping. In-situ stress measurements might be made in areas where more information is needed to interpret pulse velocity measurements, or in locations

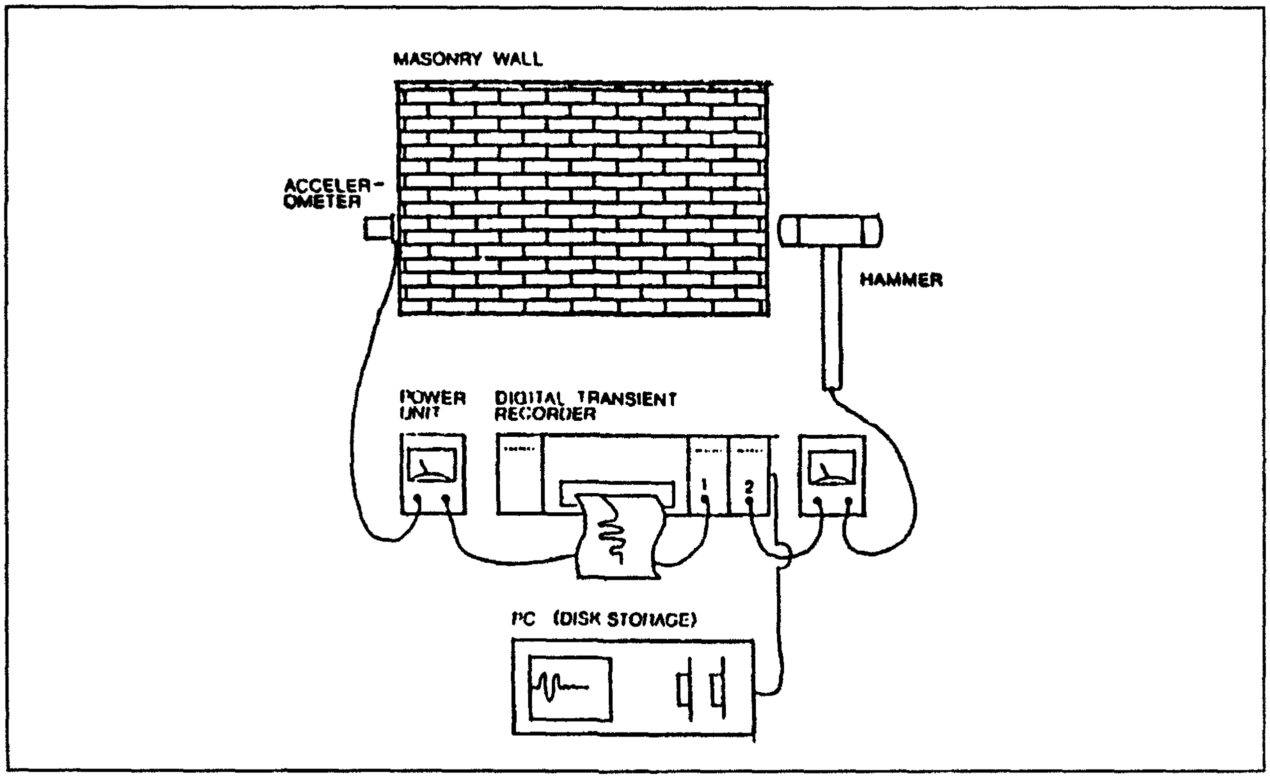

Figure 5. Mechanical pulse testing apparatus

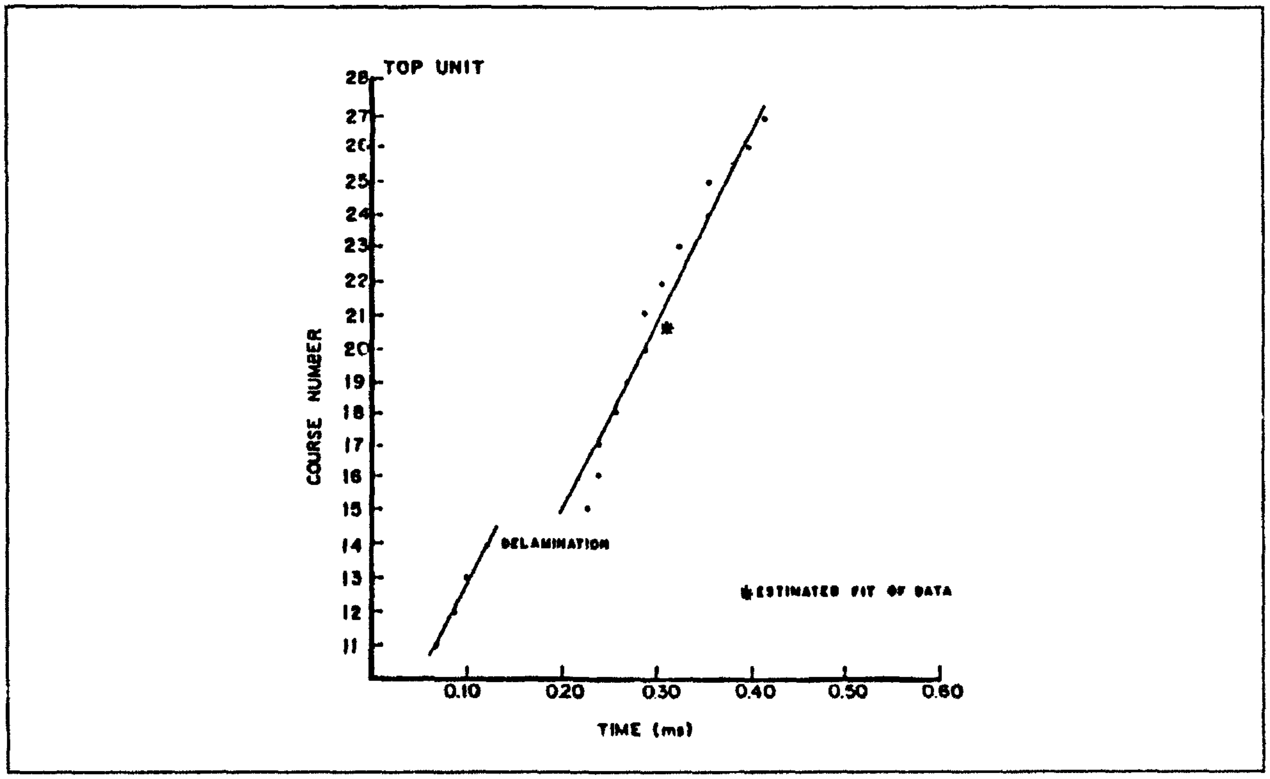

Figure 6. Effect of delaminated bedjoint on mechanical pulse velocity

defined by building analysis needs. In general, the procedure and methods used would vary depending on individual building requirements. In all cases, considerable experience and judgment would be required for the accurate interpretation of results.

Table 1 summarizes information that can be obtained by using the various NDE methods discussed in the paper. Table 2 discusses the advantages and disadvantages of the different techniques.

Table 1
Use of NDE Methods

Required Information for Structural Evaluation	Schmidt Hammer	Single Flapjack	Double Flapjack	In-Place Shear	Modified Shear Test	Ultrasonic Pulse	Mechanical Pulse	Magnetic Method	Visual
Material Properties									
Compressive Strength (Direct)			x						
Compressive Strength (Indirect)	x					+	+		
Deformability			x						
Joint Shear Strength				+	x				
Coulomb Shear Relationship					x				
Condition									
Voids Between Wythes						x	x		
Cracks in Outer Wythes						+	+		+
In-Situ Stress		x							
Material Uniformity	x					x	x		+
Location of Reinforcement								x	

Column group heading: Nondestructive Testing Techniques

x Useful for evaluation.
+ Useful, but may require additional information regarding loading conditions and crack distributions.

Acknowledgements

The author gratefully acknowledges the work of Dr. James L. Noland of Atkinson-Noland & Associates, Inc., Boulder, Colorado, from whose research much of the information in this paper is provided. Dr. Noland's firm has written a chapter on NDE methods that will be included in the revised TM 5-809-3, *Masonry Structural Design for Buildings*. Credit is also due the National Science Foundation, who funded much of the research on NDE methods in masonry.

References

Abrams, D.P. and Epperson, G.S. 1989. "Nondestructive Evaluation of Masonry Buildings." Report of Research sponsored by the Army Research Office, Washington, DC.

American Society for Testing Materials. "Standard Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry," Washington, DC.

Noland, J.L., Atkinson, R.H., and Baur, J.C. 1982. "An Investigation into Methods of Nondestructive Evaluation of Masonry Structures." Report to the National Science Foundation, Atkinson-Noland and Associates, Boulder, CO.

Noland, J.L., Atkinson, R.H., and Schuller, M.P. 1990. "A Review of the Flatjack Method for Nondestructive Evaluation," *Proceedings, Conference on Nondestructive Evaluation of Civil Structures and Materials*, Boulder, CO,

Noland, J.L., Atkinson, R.H., Kingsley, G.R., and Schuller, M.P. 1991. "Nondestructive Evaluation of Masonry Structures," Report to the National Science Foundation, Atkinson-Noland and Associates, Boulder, CO.

Table 2
Comparison of NDE Techniques

Advantages	Disadvantages
Schmidt Hammer	
Simple to use Establishes uniformity of properties Equipment is inexpensive and readily available	Evaluates only the local point and layer (wrythe or leaf) of masonry to which it is applied No direct relationship to strength or deformation properties Unreliable for the detection of flaws
Single Flatjack In-situ Stress Test	
Can establish the state of compressive stress, in-situ, with reasonable accuracy Inexpensive materials and equipment Uncomplicated to use ASTM standards currently being developed	Somewhat time-consuming to prepare the test, when compared to other methods Requires removal of mortar from masonry bed joint with a saw or drill Requires repair of the mortar joint after testing
Double Flatjack In-situ Deformability Test	
Can establish deformation properties, in-situ, with reasonable accuracy Inexpensive materials and equipment Uncomplicated to use ASTM standards currently being developed	Somewhat time-consuming to prepare the test, when compared to other methods Requires removal of mortar from masonry bed joint with a saw or drill Requires repair of the mortar joint after testing
In-Place Shear Test	
Can establish joint shear strength in-situ Equipment is inexpensive and readily available Uncomplicated to use	Somewhat time consuming to prepare Requires removal of a masonry unit and a head joint Restricted to masonry with low cement-content mortar Requires unit and mortar replacement after the test State of compressive stress on the test unit must be estimated Contribution of the collar joint is unknown
Two Flatjack Modified In-Place Shear Test	
Can establish the joint shear strength in-situ with reasonable accuracy Permits control of compressive stress on test unit Determines the Coulomb failure surface for the material	Somewhat time-consuming to prepare Requires removal of two masonry units Restricted to masonry with low cement-content mortar Requires unit replacement after the test Contribution of collar joint is unkown Requires revoval and replacement of two mortar joints Large amount of equipment is required
Ultrasonic Pulse Velocity	
Simple to use Establishes uniformity of properties Can detect flaws, cracks, or voids Possible to record trace of stress wave for analysis Equipment readily available and only moderately expensive Equipment package is self contained and portable	Requires access to both sides of a wall for direct measurements Attenuation of signal in older or soft masonry restricts distance between transducers for indirest and semi-direct use Coupling material needed between masonry and transducers, which may alter the appearance of the masonry Grinding may be required to prepare a rough surface No direct correlation with material properties
Mechanical Pulse Velocity	
Reasonably simple to use Establishes uniformity of properties Can detect flaws, cracks, and voids Possible to record trace of stress wave for later analysis Equipment readily available and only moderately expensive Capable of testing over long distances in any type of masonry Does not damage the masonry	Several pieces of equipment are involved, not easily portable Requires a separate instrument to record the wave arrival time No direct correlation between results and material properties Analysis of the wave trace can be complicated
Magnetic Methods	
Equipment is portable and inexpensive Large areas of masonry can be quickly evaluated Accurately maps location and orientation of reinforcing steel in masonry Can be used to locate metal ties and connectors	Readings can be ambiguous, requireing operator interpretation of destructive tests to verify conclusions Misidentification of metal conduit, etc., as reinforcing steel is possible Accuracy in determination of bar size and depth is questionable

Dynamic Testing for Design of a Reinforced Concrete Radar System Facility

by

Joseph M. Serena III,[1] Arthur Dohrman, PE,[2] and William H. Zehrt, Jr.[1]

Abstract

The Ground-Based Radar-Experimental Facility is the test bed for a state-of-the-art radar system to be used as part of the Nation's Strategic Defense System. This prototype is to be built and tested at the Kwajalein Atoll, Republic of the Marshall Islands. The facility includes construction of a cylindrical reinforced concrete tower, 50 ft in diameter by 115 ft tall. The tower is located inside and extends through the roof of an existing massive concrete building. The tower will support the gravity loads of the system, including the 1.1 million pound radar turret. The existing building will provide lateral stability against wind and seismic loads. To ensure adequate performance of the radar system, the combined facility is required to meet specific dynamic vibration criteria. This was ensured using a combination of computer analysis and actual structural and geotechnical dynamic testing at the existing building. This paper discusses the dynamic stiffness requirements, the dynamic testing program, and the use of test results to verify computer analysis and design.

Introduction

The Ground-Based Radar-Experimental (GBR-X) program is a research and development project being performed by the US Army Strategic Defense Command (USASDC). The program includes the design, construction, installation, and testing of a very large phased-array radar system. The purpose of the project is to validate the technology for use in identification, discrimination, and tracking of reentry vehicles in the midcourse and terminal flight phases as part of the Nation's Strategic Defense System. Design and demonstration of the GBR-X system is being performed by the Raytheon Company of Wayland, MA. As one of Huntsville Division's primary missions in support of USASDC, we provided design services for the GBR-X radar system facilities. Construction of the facility was to be performed by the US Army Engineer District, Honolulu. The site chosen by USASDC was Kwajalein Island, at Kwajalein Atoll in the Republic of the Marshall Islands, home of the US Army Kwajalein Missile Range. The GBR-X system was sited in an existing reinforced concrete structure. In the final radar design concept, the stiffness and vibrational characteristics of the entire structural system became a critical concern. There were no data available to verify the stiffness of the existing facility and its contribution to the overall

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dynamic performance of the GBR-X system. It became clear that testing was required.

To validate the facility design, two dynamic test programs were carried out. The first was a structural vibration test of the existing building. The second was geotechnical testing to determine specific properties of the soil under the building. This paper summarizes the test methods and results of the two test programs. The use of these results in modeling the total structural system and validating the facility design is also presented.

GBR-X Program Facility Overview

The principal component of the GBR-X radar system is a large turret structure which contains much of the radar system hardware. This turret is approximately 60 ft tall, 35 ft square in plan, and weighs 1.1 million pounds. The turret is capable of rotating through 356 deg in the horizontal plane and elevating to point up to 75 deg from horizontal. These motions impose significant inertial live loads on the supporting structure. The GBR-X turret is illustrated in Figure 1.

The facility chosen to house the GBR-X system is the Defense Control Center Building (DCCB) at Kwajalein. This building is a massive reinforced concrete structure, originally intended for use in the Safeguard anti-ballistic missile program. The DCCB is essentially an irregular, multifaceted shell of walls ranging in thickness from 12 to 36 in., with an 18-in. concrete roof supported by large concrete beams and a single, central concrete column. Interior floors in the DCCB are steel or composite decking supported on steel beams and columns that are independent from the concrete walls. The low roof of the DCCB is 46 ft above grade, and the high roof is 110 ft above grade.

Although a massive structure, the DCCB roof is not sufficiently strong to support the large dead and live loads of the GBR-X radar turret. Therefore, the final design included the construction of a turret support tower inside the building. This tower is a 50-ft-diameter concrete cylinder, with a wall thickness of 12 in., extending from the ground level through the DCCB roof. Total height of the tower is 115 ft. The tower will provide direct support for the turret and will support several internal floors housing computers and offices. The tower is tied to the DCCB roof to provide lateral stability to the system. However, the tower is designed to pick up no dead load from the DCCB roof. Also, the interior floors are connected to the tower with sliding connections so that they will not add lateral vibrations to the tower. A cross section of the DCCB/tower design is shown in Figure 2.

Structural Vibration Requirements

The final evolution of the GBR-X design imposed specific criteria for dynamic stiffness on the overall structural system. To meet radar performance criteria, the overall facility, including the DCCB, turret support tower, and turret, was required to have a fundamental frequency of vibration of at least 3.5 Hz. Meeting this requirement was a complicated task because of the interfaces between the different elements of the system. Huntsville Division (CEHND) was responsible for design of the renovations to the DCCB, including the turret support tower and its foundation. However, Raytheon was responsible for the design of the radar system and turret and for overall system performance. While it would be relatively easy to design an all-new facility to meet this requirement, it was impossible to guarantee, through computation alone, the dynamic performance of the hybrid of the existing DCCB and the new tower and turret structures. The only way to accurately predict the stiffness contribution of the DCCB was through dynamic testing. Data from the tests could then be included in a rational analysis of the entire structural system.

Structural Dynamic Testing

The scope of the structural vibration tests was to perform a combination of forced vibration and ambient vibration measurements on the DCCB. These measurements would provide baseline information on the vibrational

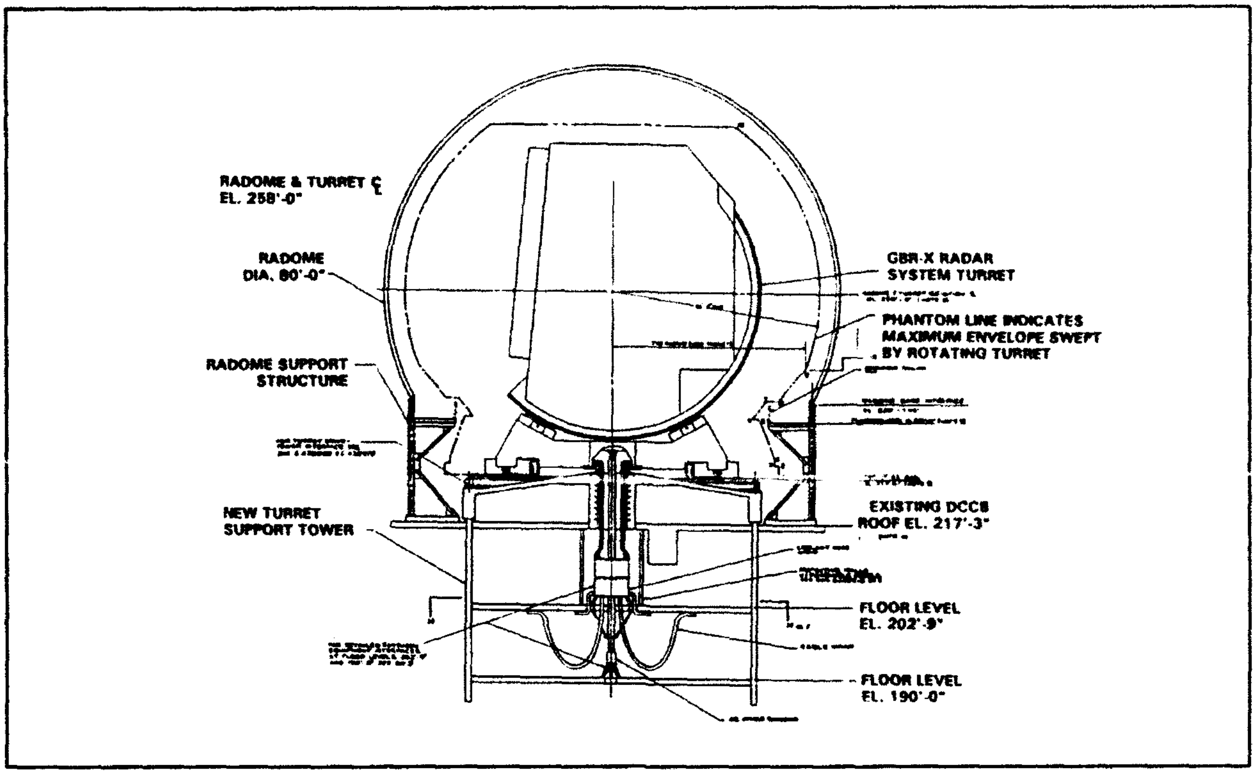

Figure 1. GBR-X radar turret and radome

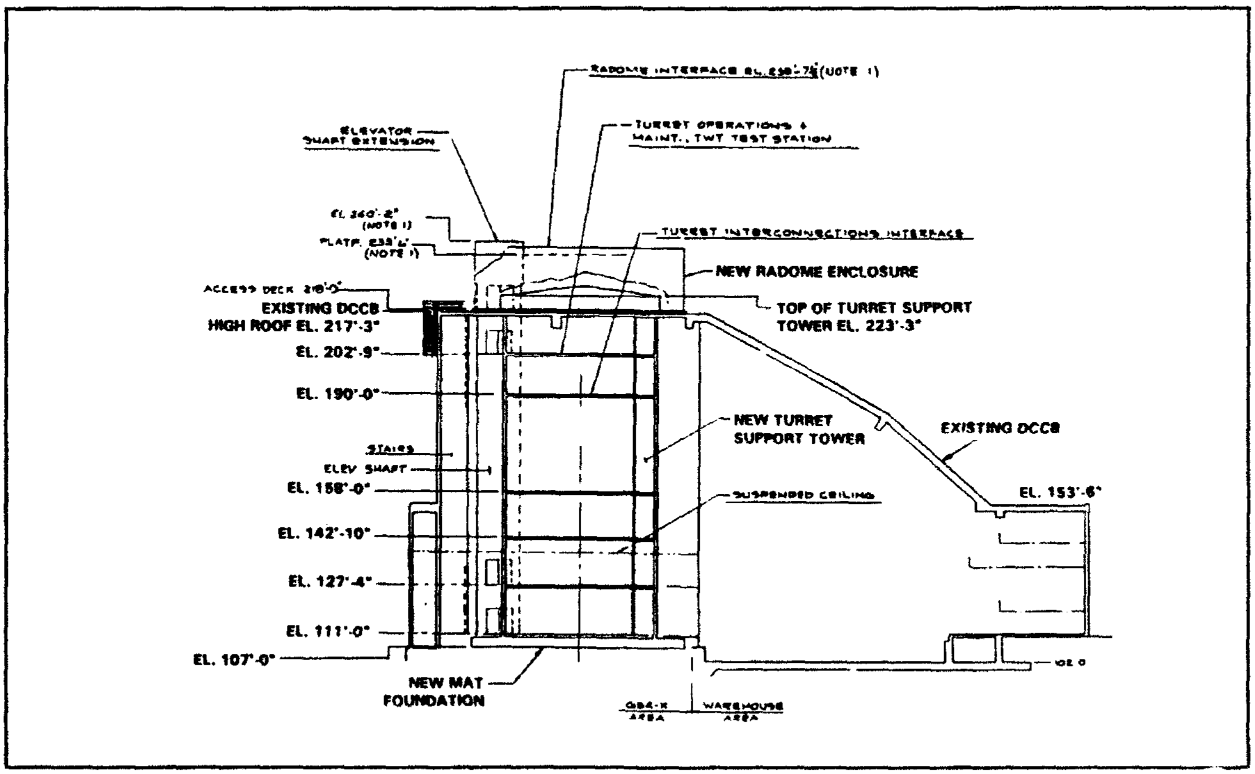

Figure 2. DCCB and turret support tower

characteristics of the DCCB. Specifically, frequencies and approximate mode shapes were desired for the fundamental modes of vibration about the two principal axes of the structure, essentially the north-south and east-west directions. Additionally, a prediction of the dynamic response of the building to ambient wind loads was desired. Dynamic testing was performed by Failure Analysis Associates, Inc., of Palo Alto, CA, under subcontract to Raytheon.

Test Methods and Equipment

A hydraulic actuator/rolling mass system was used to generate steady-state lateral forces at the roof of the DCCB. This system is depicted in Figure 3. The rolling mass used was a large concrete block mounted in a steel carriage on rollers. The total weight of the mass was 5,800 lb. The mass was attached to a hydraulic actuator and a 20,000-lb load cell, in series, which was in turn attached to a steel reaction bracket. The steel bracket was bolted to a steel plate epoxied to the DCCB roof at the center of the proposed turret support tower. The hydraulic actuator was programmed to extend and retract cyclically and was controlled through the use of pressure limit switches. This system allowed direct force vibration inputs over the range of approximately 0.6 Hz to 7.0 Hz. The applied vibration was not purely sinusoidal but included harmonics at frequencies which are integer multiples of the basic applied frequency. Through these harmonics, it was possible to provide strong excitation at frequencies in excess of 40 Hz. The load cell was used to record real-time forces applied to the DCCB roof.

Vibration response was monitored using accelerometers acting over a range of 0-30 Hz. A total of 13 accelerometers were used. These were located to obtain lateral (X and Y) and vertical (Z) responses at various locations on the high roof, at a point 20 ft below the high roof, on the low roof, and at the building foundations. One accelerometer was attached to the rolling mass carriage to record the amplitude of the applied displacements. The signals from the accelerometers were amplified and filtered and were displayed on an oscilloscope during testing. The signals were recorded on a tape recorder for backup and later processing. Real-time data collection and processing was accomplished on-site using a portable computer equipped with data acquisition hardware and software.

For processing the test data, the DCCB was modeled as a single-degree-of-freedom system. For this model, the theoretical displacement response depends on the amplitude of the applied excitation force, its frequency, the natural frequency of the system, and the amount of damping. The ratio of response amplitude to force amplitude, or displacement per unit of force, is called the dynamic compliance, a , and is expressed as

$$a = \frac{x}{f} = C\left\{\left[1 - \left(\frac{w}{w_o}\right)^2\right]^2 + \left(\frac{2cw}{w_o}\right)^2\right\}^{-1/2}$$

where

w_o = natural frequency

w = excitation frequency

c = damping ratio

x = relative displacement amplitude

f = force amplitude

C = static compliance (essentially, the static flexibility)

For lightly damped structures, the dynamic compliance rapidly increases as the forcing frequency approaches the system fundamental frequency, where the dynamic response is amplified by a factor of $1/2c$ above the static response. Below the fundamental natural frequency, the dynamic compliance approaches the static compliance. At frequencies above the fundamental, the dynamic compliance will increase and decrease depending on the separation between the excitation frequency and any higher-mode natural frequencies. By

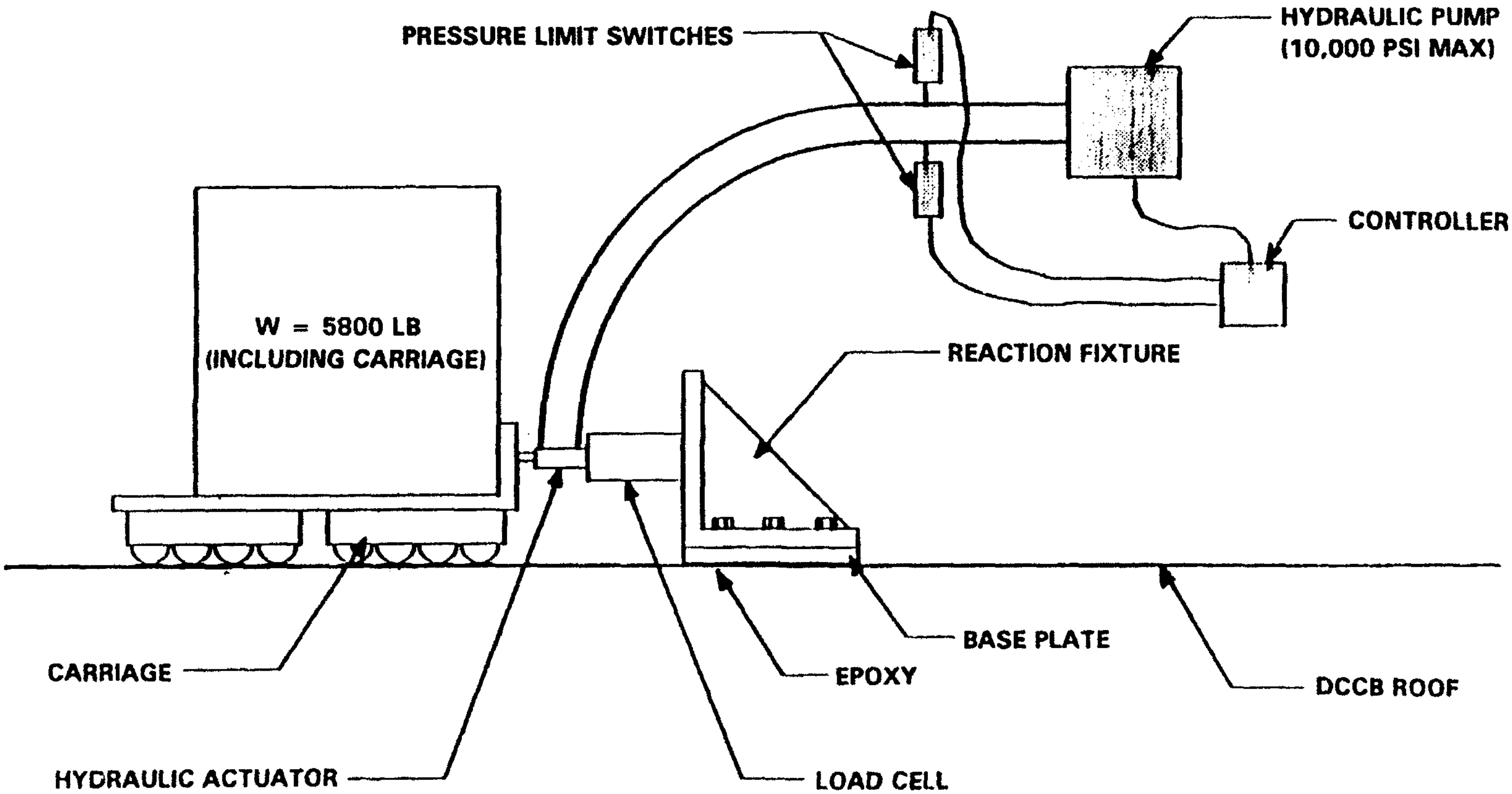

Figure 3. Hydraulic Actuator/rolling mass excitation system

measuring the excitation force and response over a wide range of frequencies, one can determine the frequency corresponding to maximum response and thus identify the natural frequency. Given the natural frequency, the effective modal mass can be obtained.

Vibration tests were performed in both the north-south and east-west directions. The natural frequencies were roughly identified by running sweep tests through a wide range of frequencies and watching the response data from accelerometers on the oscilloscope. The visually identified peak deflections allowed narrowing of the test band of vibration frequencies around the peaks to gather specific force and response data. Given the force and deflection data, the natural frequency, damping, and static compliance were estimated using a least squares curve fitting technique. Dynamic compliance was plotted versus frequency for the data collected, and the natural frequencies were identified. One such plot for the north-south direction tests is shown in Figure 4.

Test Results

Results of the vibration tests are given in Table 1. The natural frequency of vibration in the north-south direction was identified as 4.5 Hz. The corresponding mode shape is shown in Figure 5. In the east-west tests, two resonant frequencies were discovered at 5.0 Hz and 5.3 Hz. Examination of the accelerometer data and comparison to the north-south data lead to the conclusion the east-west frequency is 5.3 Hz, with a corresponding mode shape similar to that for the north-south mode. The 5.0 Hz vibration appears to be the fundamental torsional mode of the structure. The test data also permitted calculation of the dynamic compliance for each mode. The a_{95} value is the upper bound for the dynamic compliance to a confidence level of 95 percent. That is, there is 95-percent certainty that the actual dynamic compliance for a given mode will be less than the a_{95} value.

It should be noted that the best least-squares fit of the data produced a damping ratio of 6.4 percent. This is somewhat higher than the 3-5 percent that would be expected for a concrete structure. The extra damping was assumed to be an artifact of the energy losses in the hydraulic shaker system due to impacting and sliding of the mass and rollers. Calculations for damping ratios of 3 to 6.4 percent showed that there is very little variation in the natural frequency with variation in damping. However, larger damping values resulted in larger values of static and dynamic compliance. For purposes of this test program, a damping value of 4 percent was assumed for computing the final natural frequencies.

Table 1
Structural Vibration Test Results

Mode	Frequency (Hertz)	Dynamic Compliance (in./lb)	a_{95} Dynamic Compliance (in./lb)	Modal Mass (lb)
N-S	4.5	5.17×10^{-8}	9.04×10^{-8}	9,300,000
E-W	5.3	3.20×10^{-8}	8.48 x 10-8	10,900,000
Torsion	5.0	1.64×10^{-8}		

The response of the DCCB to ambient excitation by the wind was monitored at various intervals during the direct vibration tests. Wind speed was monitored with a single anemometer. The wind speeds during the tests were 20-25 mph, with some gusts up to 30 mph. The vibration response to the wind excitation was extremely small. The root-mean-square amplitude of vibration was 1.2×10^{-5} in. at 26.5 mph. The relationship between wind speed and vibration amplitude was determined to be

$$X_{RMS} = 1.7 \times 10^{-8} v^2$$

where v^2 is the average wind speed. Accelerometer data revealed that the primary mode excited by the ambient winds is the north-south fundamental. Acceleration at this frequency is approximately 3×10^{-6} G. Acceleration at the DCCB foundations due to wind was below the noise threshold of the instrumentation. Therefore, the acceleration at the DCCB foundation caused by the wind is less than 8×10^{-7} G.

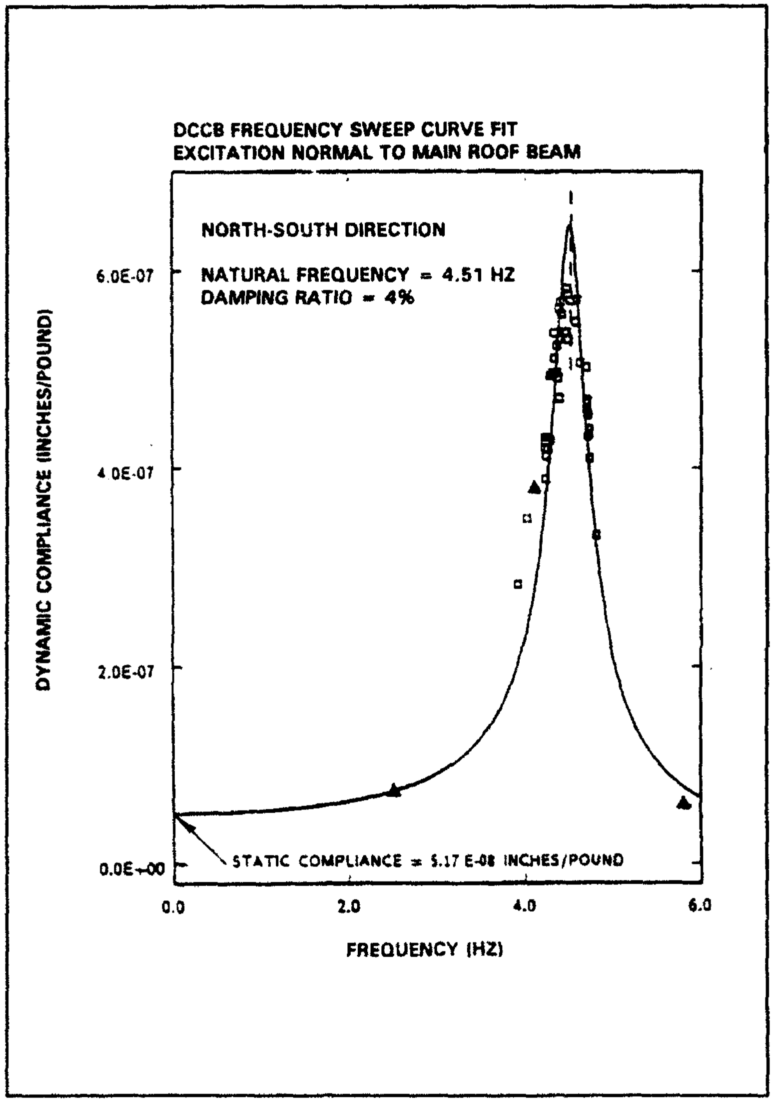

Figure 4. Typical frequency sweep curve fit plot

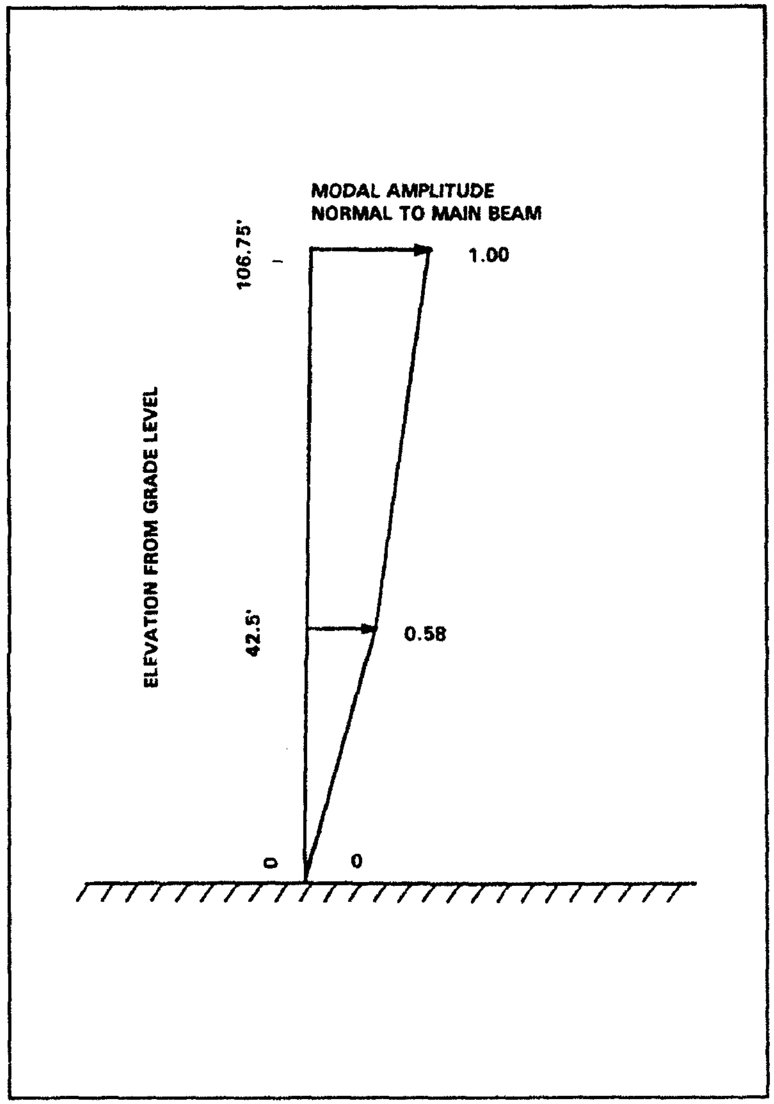

Figure 5. DCCB north-south fundamental mode shape

Use of Structural Test Data in Analysis

The data from the structural vibration tests were used by Raytheon to model the overall structural system. Specifically, the dynamic compliance and modal mass data were used by Raytheon and its structural consultants to model the contribution of the DCCB existing structure. This effect was modeled as a spring-mass support system at the DCCB roof elevation, as part of a finite element model that included the stiffness of the turret support tower and the turret elevation and rotation mechanisms. This model is shown schematically in Figure 6. Analysis of the model predicted that the natural frequency of the overall system will be 3.65 Hz, which exceeds the 3.5 Hz minimum frequency requirement. The use of the 95-percent dynamic compliance values lends a high degree of certainty to this prediction. Statistically, there is 95-percent certainty that the actual compliance will fall below the a_{95} values. That is, the DCCB will almost certainly be less flexible than predicted. Therefore, the actual natural frequency of the combined structural system will almost certainly be greater than the 3.65 Hz prediction.

Geotechnical Testing

One important element of the total system stiffness was the stiffness of the foundation. For a given foundation configuration, the dynamic foundation stiffness is dependent on the dynamic shear modulus G of the soil beneath the foundation. Due to the space constraints inside the existing building, the foundation configuration was relatively fixed. In modeling the structure, the consultant had assumed a value of 10,000 psi for G. This was based on dynamic tests performed on another island at Kwajalein Atoll for a different radar system. Using this value, the foundation was amply stiff. Unfortunately, data were not available as to the actual value of G at the proposed building site.

We followed a two-track approach toward reducing this uncertainty. At our request, Raytheon's structural consultant performed a sensitivity study of the effect of varying soil shear modulus on fundamental frequency of the tower. The results of this study indicated that for values of dynamic shear modulus of 3,000 psi or below, the fundamental frequency of the system would be governed by the vertical motion of the foundation. This would have led to unacceptable vibrations in the structure. Concurrently, we undertook in situ measurements of the actual dynamic characteristics of the soil.

Pacific Ocean Division and Waterways Experiment Station (CEWES) performed the field work. Several geophysical methods were used in an effort to correlate the results. The primary method used was the crosshole seismic technique. This method measures the time required for vibrations to travel though the soil from one borehole to an adjacent borehole. The shear wave velocity can be used to calculate a value for G. Surface seismic refraction surveys were also performed. These measure travel time between a vibration source and an array of geophones, all placed on the ground surface. From the travel times at the different locations, the wave velocities and also the soil layering can be determined. These methods are illustrated in Figure 7. The seismic methods have the advantage of being representative of the entire soil mass between vibration source and sensors, as opposed to laboratory tests on a small sample, which may not be representative of the entire soil mass. We also attempted to measure soil stiffness directly by means of a plate bearing test. This attempt was not completely successful.

Three 60-ft-deep borings were drilled adjacent to the existing building in an area which had been subjected to the same loading history as the area under the building. Figure 8 is a plan view of the test locations. After the holes were cased, a borehole deviation survey was performed. This procedure is used to determine the exact distance between points in adjacent borings, a necessary input in figuring wave velocity. Velocity testing was then carried out in two configurations: crosshole, where the vibration source and the receiver are at the same

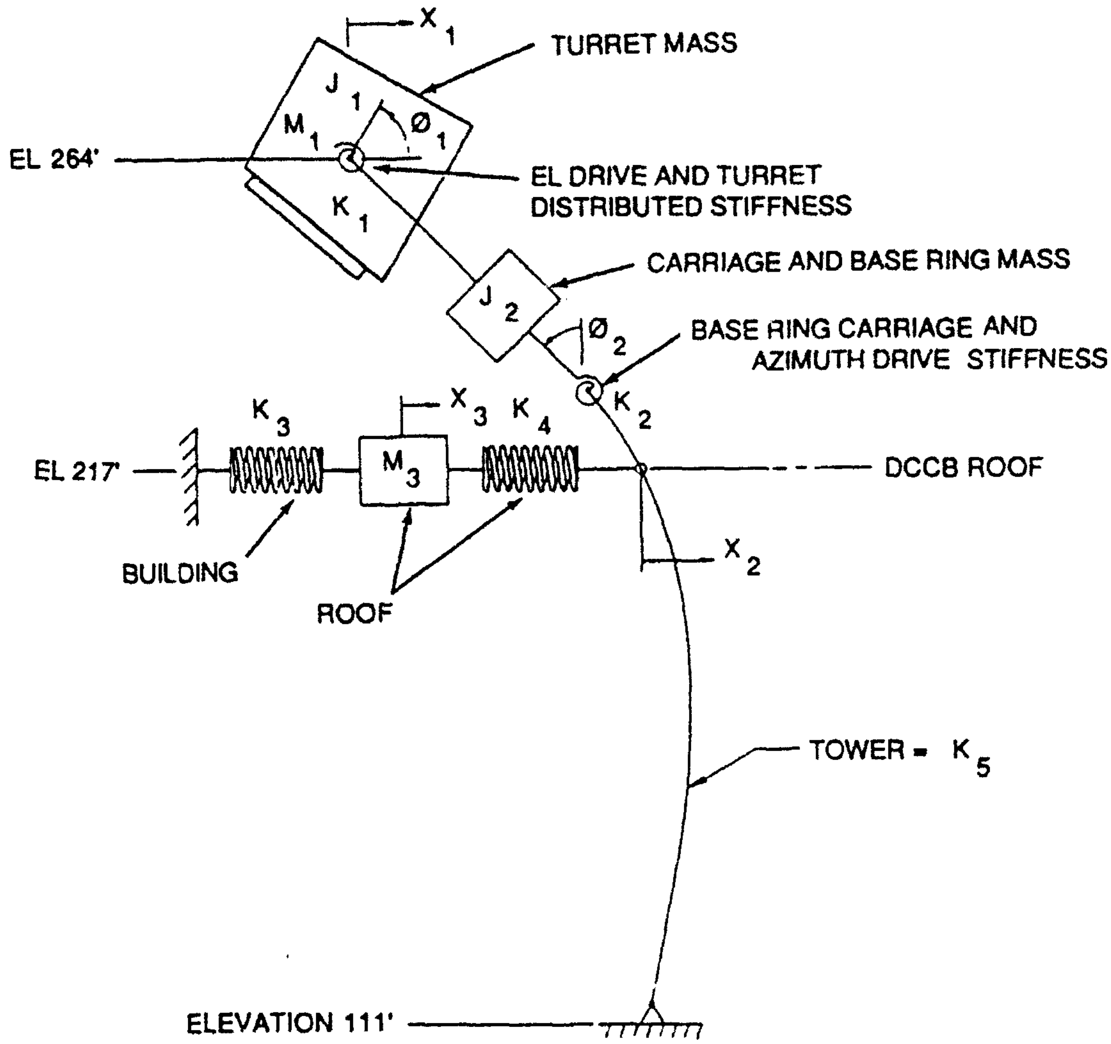

Figure 6. Simplified GBR-X structural system finite element model

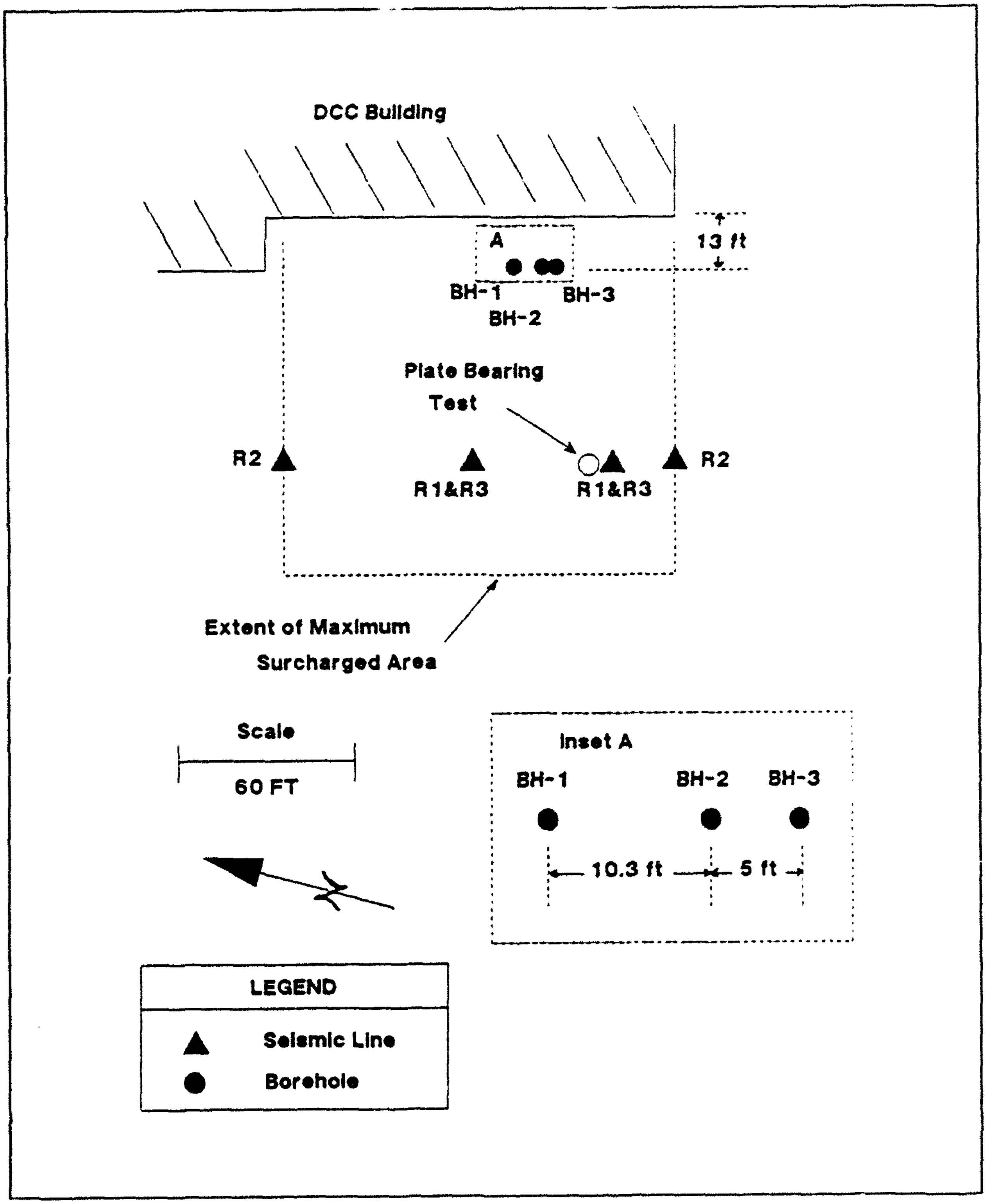

Figure 7. Geotechnical seismic survey methods

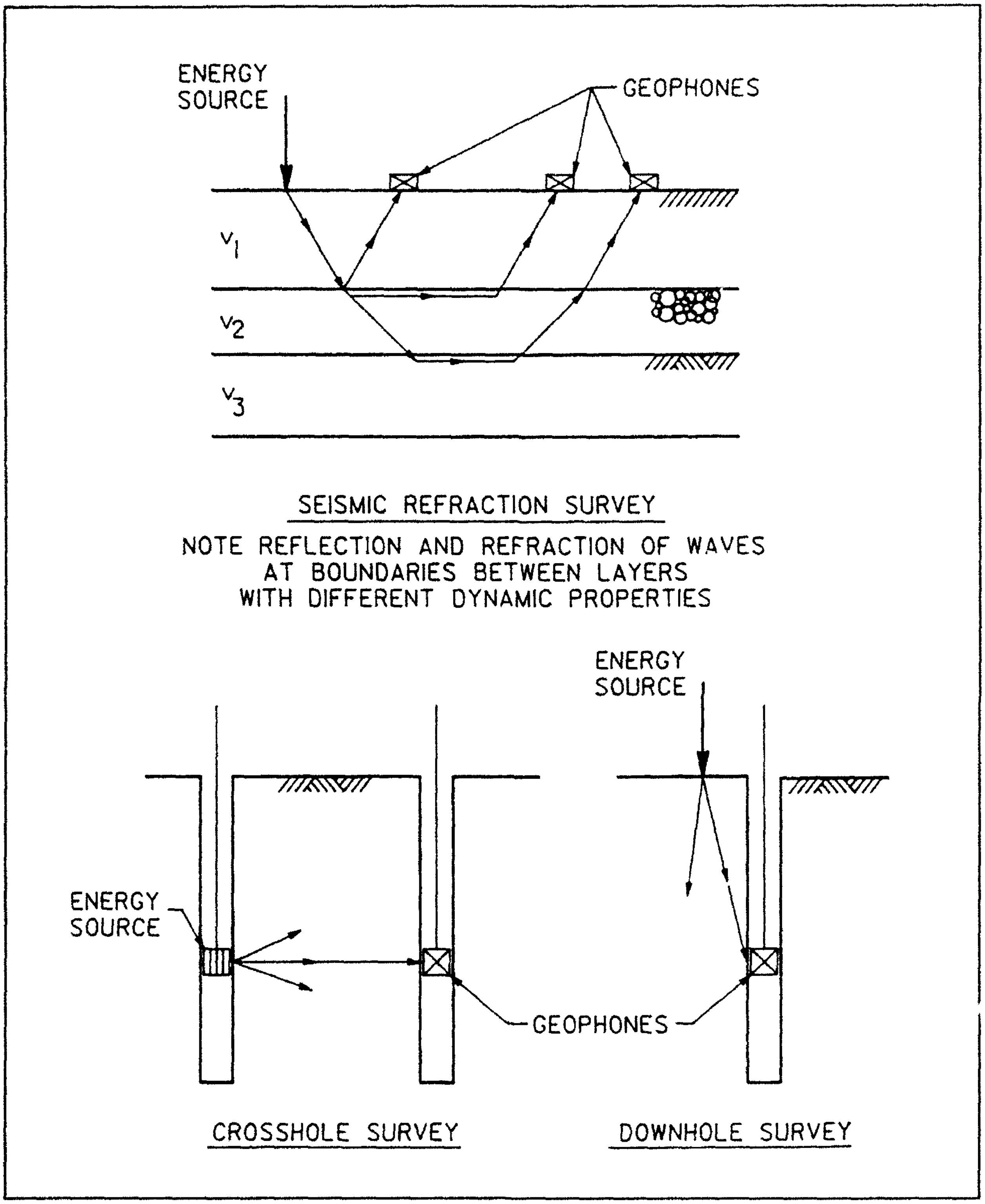

Figure 8. Soil survey test layout, plan view

depth in each boring, and downhole, where the vibration source is kept at the surface while the receiver is lowered into the hole. Tests were conducted for compression wave and for shear wave velocities. Measurements were conducted at 2.5-ft-vertical intervals for the crosshole shear wave tests, and 5-ft-vertical intervals for all other tests.

The surface seismic refraction tests, conducted in the same general area, tended to confirm the results of the crosshole and downhole tests in terms of wave velocities and layering.

The results of these tests indicated that the soil could be idealized in terms of four layers. From the surface to a depth of 7.5 ft, the compression wave velocity was approximately 1,500 fps and the shear wave velocity was approximately 600 fps. Using the relation

$$G = \rho * v_s^2$$

where

v_s = shear wave velocity

ρ = mass density of the soil

G = for this layer was calculated to be 7,700 psi.

Poisson's ratio was calculated using the two relations

$$v_r = \frac{v_p}{v_s}$$

$$\nu = \frac{v_r^2 - 2}{2(v_r^2 - 1)}$$

where v_p = compression wave velocity and ν = Poisson's ratio. Poisson's ratio was calculated to be 0.405. Finally, knowing the shear modulus and Poisson's ratio permitted the calculation of Young's modulus E by the relation

$$E = 2(1 + \nu)G$$

Young's modulus for the surface layer was determined to be approximately 22,000 psi. Similarly, the next layer, from 7.5 to 20 ft was determined to have a G of 6,700 psi, a nu of 0.495, and an E of 20,000 psi. The third layer, from 20 to 40 ft, was determined to have a G of 10,000 psi, a ν of 0.495, and an E of 31,700 psi. The fourth layer, from 40 to 60 ft, was determined to have a G of 18,000 psi, a ν of 0.492, and an E of approximately 53,000 psi.

CEWES staff members also used the method proposed by Seed and Idriss (1970) for determination of an empirical parameter K_2 to relate shear modulus G to effective mean confining stress σ_m'. In this method, knowledge of the confining stress and shear modulus at a number of points allows one to calculate the value of K_2 for each soil layer. This permits the estimation of shear modulus for other points in the layer. The formula proposed by Seed and Idriss is

$$G = 1000 * K_2 (\sigma_m')^{1/2}$$

where σ_m' is the average of the vertical and horizontal effective stresses. The K_2 values calculated at the GBR-X site ranged from 31 to 89. The design values recommended for K_2 were 68 for the uppermost soil layer from 0 to 7.5 ft, 33 for the second layer from 7.5 to 20 ft, 42 for the third layer from 20 to 37.5 ft, 48 for the fourth layer from 37.5 to 50 ft, 55 for the fifth layer from 50 to 55 ft, and 65 for the sixth layer from 55 to 60 ft. Although we did not need to use K_2 to estimate shear modulus for this project, had the project site been moved out of the immediate area we could have used K_2 to estimate values for shear modulus at the new site. The design values selected for K_2 and the modulus values are shown in Figure 9.

The last in situ test performed was a plate bearing test. This consisted of static and cy-

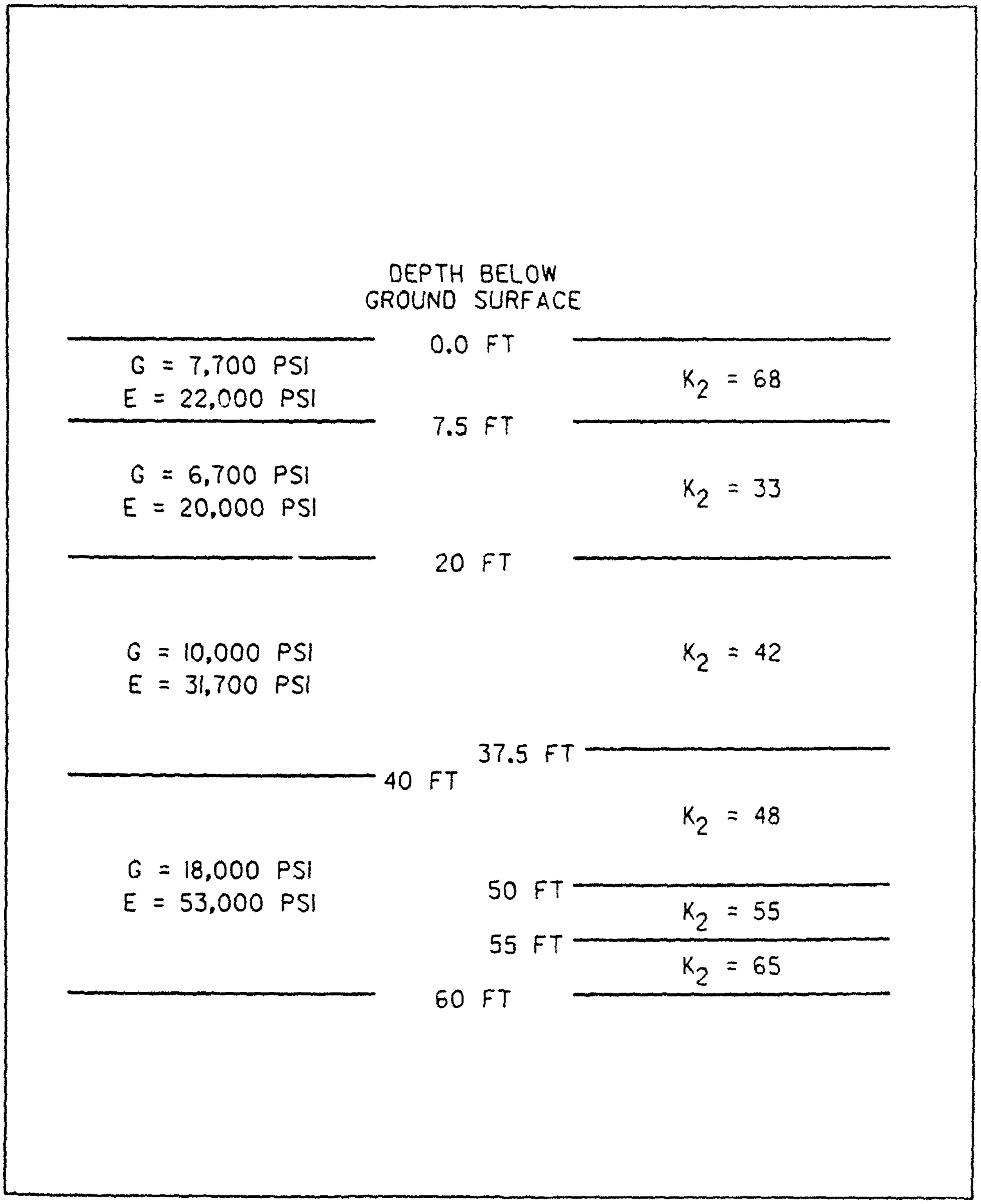

Figure 9. Geotechnical test results

clic loads imposed on a 26.6-in. square steel plate. From this test, a Young's modulus E of 123,600 psi was determined. This does not correlate well with the seismic methods or the K_2 parameter method. One possible explanation might be that the plate bearing test measures the properties of a relatively small mass of soil and, at this location, happened to fall on a denser, stiffer pocket of soil.

Of the in situ soils tests performed, the seismic tests provided the most information on the site conditions. Refraction methods have the advantage of providing information on layering as well as wave velocities without requiring borings. Crosshole and downhole methods, because borings are required, allow the collection and examination of soil samples and permit the direct measurement of soil properties in each layer. Calculation of K_2 values from the tests performed at one location gives one the flexibility to estimate dynamic soil properties at a slightly different location, without the expense of additional dynamic testing.

Summary

The overall dynamic stiffness of the DCCB/tower/turret structural system is critical to the performance of the GBR-X radar system. The uncertainty of the stiffness contribution of the DCCB led to the requirements for actual dynamic structural testing. The results of these tests, along with other design data developed by CEHND, were used by Raytheon and its consultants to prove that the total system would provide adequate stiffness. Foundation stiffness for the tower was an important element of the total system stiffness. The geotechnical testing at the site verified that the tower foundation design would provide acceptable performance.

The structural and geotechnical testing programs discussed in this paper were crucial to ensuring the quality of not only the GBR-X facility but that of the entire radar system. Through these test programs, the Huntsville Division was able to assist both our customer and the radar system designers, thereby contributing to the assurance of total design quality for the GBR-X program.

References

Das, B. M. 1983. *Fundamentals of Soil Dynamics*, Elsevier Science Publishing Co. Inc., New York, NY,.

Raytheon Company Report. 20 October 1989. "Ground Based Radar - Experimental Facilities Requirements Document," Contract No. DASG60-87-C-0014, Prepared for US Army Strategic Defense Command, Huntsville, AL.

Schoof, Craig C. December 1988. "Vibration Measurements of the Defense Center Control Building," Failure Analysis Associates, Inc., Palo Alto, CA, Prepared for Raytheon Company, Wayland, MA.

Seed, H. B., and Idriss, I. M. 1970. "Soil Moduli and Damping Factors for Dynamic Response Analysis," Report No. EERC 70-10, Earthquake Engineering Research Center, University of California, Berkeley.

Timoshenko, S., Young, D. H., and Weaver, W. Jr. 1974. *Vibration Problems in Engineering*, John Wiley and Sons, New York.

Yule, D. E., and Sharp, M. K. April 1990. "Foundation Investigation for Ground Based Radar Project - Kwajalein Island, Marshall Islands," US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Zarghamee, Mehdi S. 27 April 1989. "Recommended Design Changes of Turret and Tower, GBR-X Phased Array Radar," Simpson Gumpertz and Heger, Inc., Arlington, MA.

Special Seismic Design Criteria for the US Chemical Stockpile Disposal Program

by

R. Stephen Wright, PE,[1] *and Boyce L. Ross, PE*[1]

Abstract

As required by Public Law 99-145, the US Army Corps of Engineers is currently in the process of designing and constructing eight chemical weapons demilitarization facilities. The facilities will be used to destroy the US inventory of obsolete and unserviceable chemical weapons. These weapons contain extremely lethal nerve and blister agent as well as explosives. The life safety and environmental risks associated with these facilities are unprecedented for military construction and compare with those for nuclear power plants. To mitigate these risks and assure the safe operation and shutdown of these facilities in the event of earthquakes, highly specialized seismic design criteria were developed. This paper discusses the special seismic design spectra developed for each site as well as analysis procedures, facility and equipment design requirements, and specialized quality control procedures.

Program Background

Public Law 99-145, The Department of Defense Authorization Act of 1986, directed the destruction of the United States inventory of obsolete and unserviceable chemical weapons. This mission was identified as the Chemical Stockpile Disposal Program (CSDP). Alternatives considered for the CSDP included a single national disposal facility, two regional facilities, and on-site destruction. In February 1988, the Department of Defense Authorization published its Record of Decision (ROD) for CSDP in support of the Programmatic Environmental Impact Statement (PEIS) (Headquarters, Department of the Army 1988). The ROD selected onsite disposal as the safest alternative for destruction of the stockpile. Destruction of munitions at each site will involve construction of incineration facilities at eight sites within the Continental United States (CONUS) as shown in Figure 1. The facility at Tooele, UT is the first CSDP facility to be constructed within the CONUS. This facility will utilize the technology from the Johnston Atoll Chemical Agent Disposal System (JACADS). As shown in Figure 2, all CSDP operations in the CONUS are scheduled to be completed by 1999.

Present chemical stockpile

Lethal chemical agents are of two basic types (nerve and blister) and are configured in a variety of munitions and bulk containers. Bulk containers include spray tanks and ton containers. Almost 94 percent of the total stockpile is stored within the CONUS. Approximately 61

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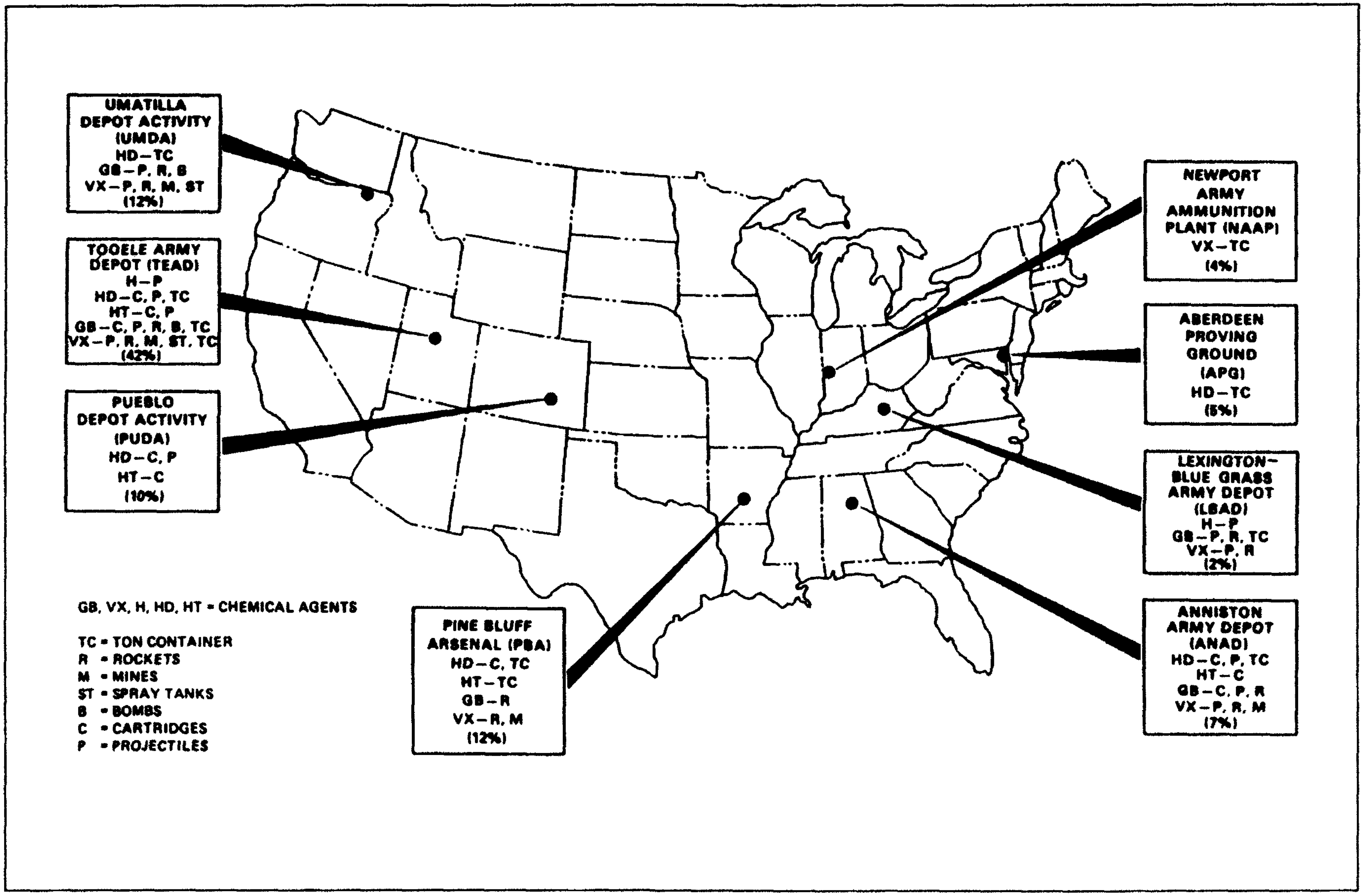
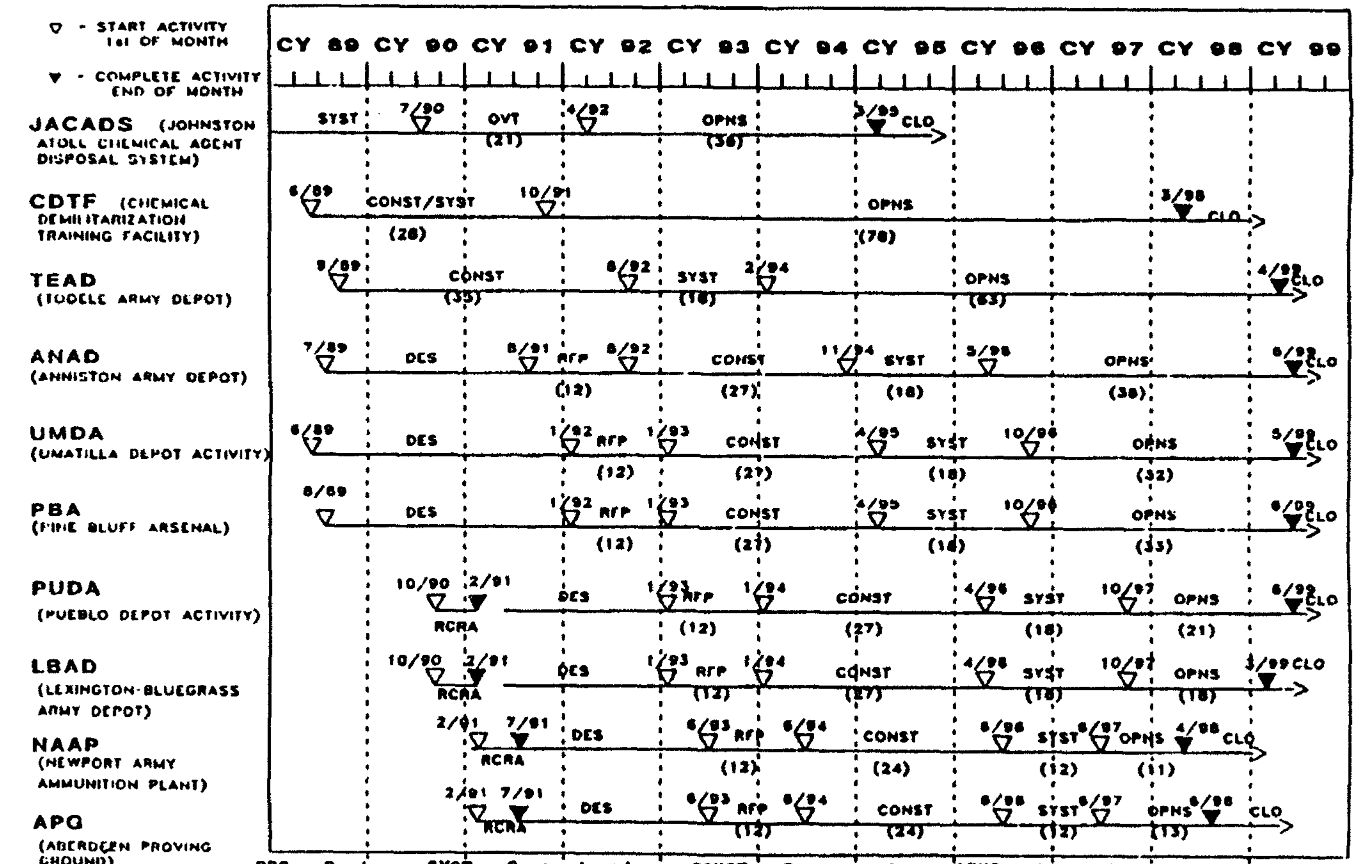


Figure 1. CONUS chemical stockpile storage sites

CHEMICAL STOCKPILE DISPOSAL PROGRAM IMPLEMENTATION SCHEDULES--REVISION 3 (MARCH 1991)



DES - Design SYST - Systemization CONST - Construction OPNS - Operations
CLO - Closure RCRA - Resource Conservation Recovery Act Permit Design Input
OVT - Operational Verification Testing RFP - Issue and Evaluate Request for Proposal

Figure 2. CSDP schedule

percent of this stockpile is stored in bulk form (ton containers or spray tanks.) Figure 1 shows the types of munitions and chemical agents stored at each CSDP site. The Newport AAP and Aberdeen Proving Grounds sites store only bulk items and will differ from the remainder of the sites since they will not be required to process items which contain explosives or propellants.

Program management

The Department of Defense Authorization assigned the overall program management of the CSDP to the Program Executive Officer, Program Manager for Chemical Demilitarization located in Edgewood, MD. Contracting for engineering services for both the process and facility designs is being provided by the Huntsville Division, Corps of Engineers (CEHND) located in Huntsville, AL. Corps of Engineer Districts will provide engineering services for support facilities within their geographic regions during construction and will be responsible for administering the construction contracts. Design services for both process and facilities have been contracted by CEHND to The Ralph M. Parsons Company, Pasadena, CA.

General Facility Description

The CSDP facilities consist of several distinct process and support facilities. Figure 3 shows the layout of the CSDP facilities.

Process facilities

The following process facilities contain or process toxic chemicals and are categorized as Essential Facilities as defined in TM 5-809-10 (Headquarters, Department of the Army 1982).

The actual disassembly and destruction of the chemical weapons occur in the Munitions Demilitarization Building (MDB). The MDB houses the basic process equipment and control systems necessary to remotely disassemble, punch, and drain munitions and bulk items; to incinerate chemical agent and other toxic liquid and solid waste; and to decontaminate munition bodies and other metal items. The MDB contains munitions processing areas used to prepare munitions for incineration. Within the munitions processing area is the Toxic Cubicle (TOX) which contains two storage tanks for agent removed from munitions: a 500-gal agent holding tank and a 1,300-gal surge tank to hold additional agent in an emergency. The greatest potential for a significant release of agent due to a seismic event originates in the TOX. The agent is then pumped to the Liquid Incinerator (LIC) for incineration. The MDB contains four incineration areas; the Deactivation Furnace (DFS) incinerates drained rockets and mines, fuzes, explosives, and propellants. The LIC incinerates drained agent and spent decontamination solutions. The Metal Parts Furnace (MPF) decontaminates drained ton containers and bulk items. Finally, the Dunnage Incinerator (DUN) incinerates both contaminated and uncontaminated dunnage.

The MDB is a two-story reinforced concrete building approximately 281 ft long by 144 ft wide. Building height is approximately 45 ft. The structural design is based on a concrete framing system with concrete load-bearing walls and columns and composite building panel siding. Lateral loads are resisted by the concrete shear walls. The first and second floor plans are shown in Figures 4 and 5, respectively.

The buildings for the Pollution Abatement System (PAS) and the DUN PAS are structural steel, braced-frame systems. The PAS and DUN PAS areas contain the equipment used to process the gaseous emissions of the incineration system to meet environmental requirements.

The Container Handling Building (CHB) provides temporary storage for chemical munitions prior to unpacking them in the MDB building. The CHB is a pre-engineered steel-frame building, except for the transition area north of the MDB building. The transition area consists of reinforced concrete walls and structural steel framing.

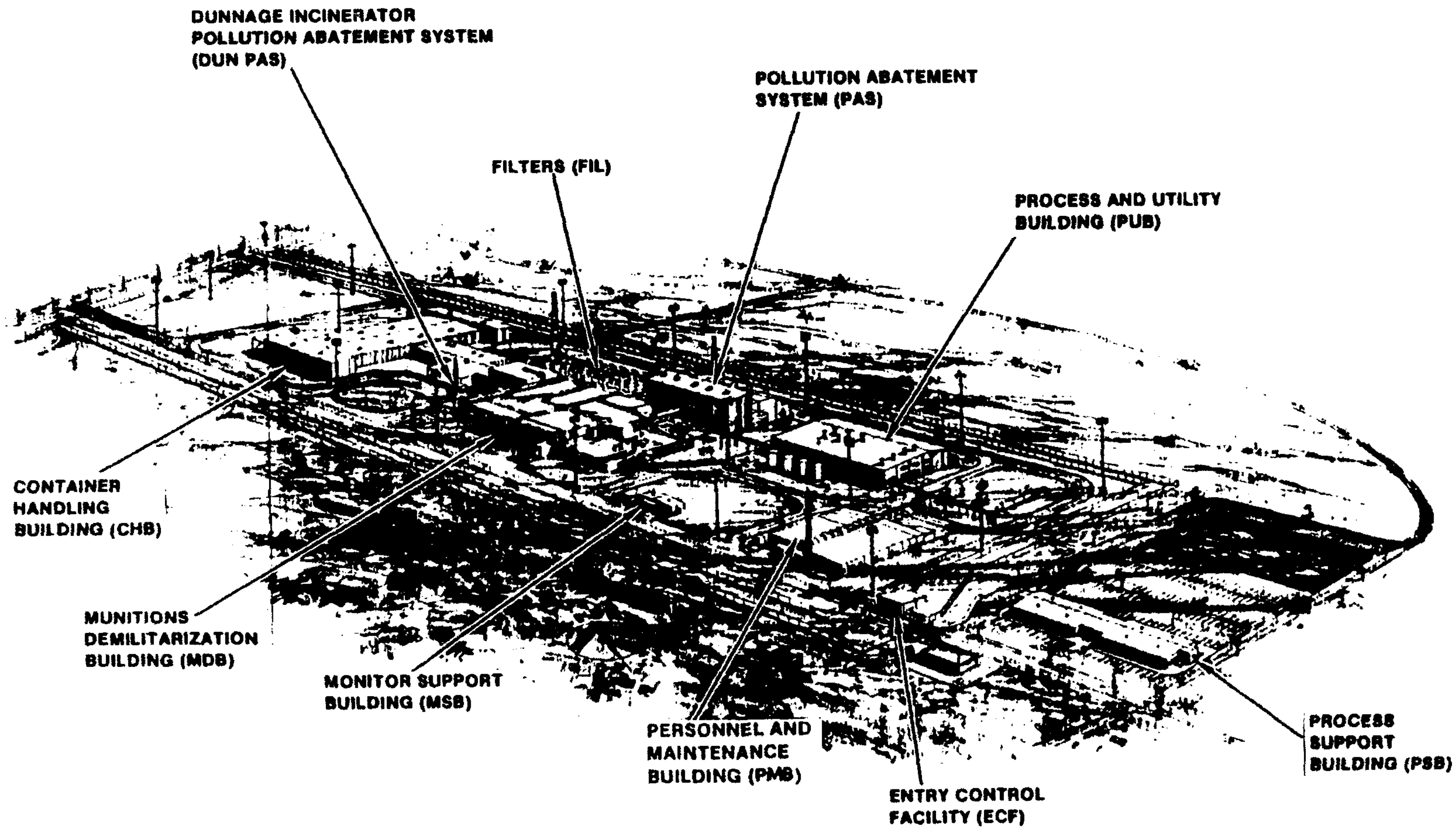


Figure 3. CSDP facility

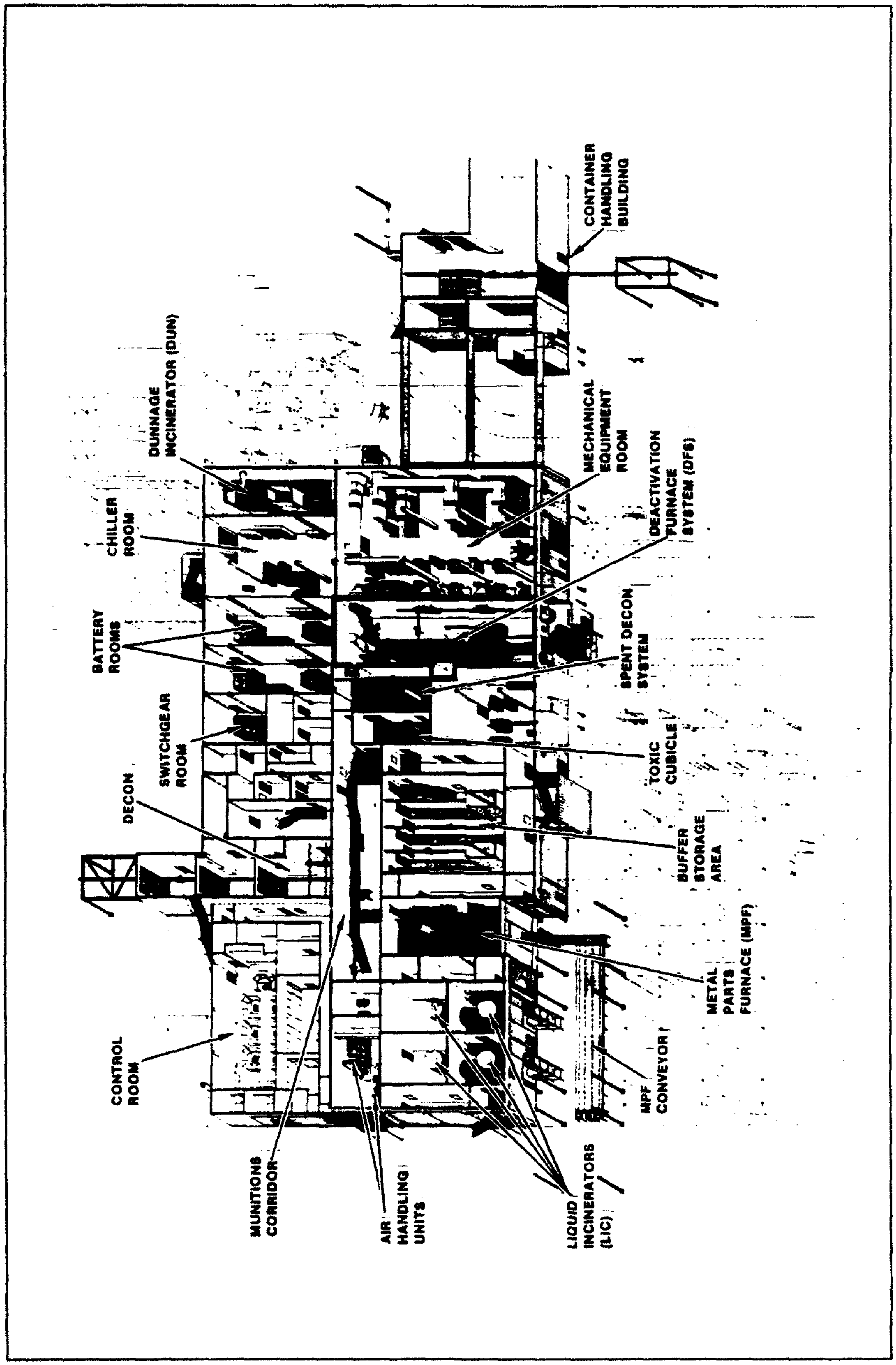


Figure 4. Munitions demilitarization building, first floor

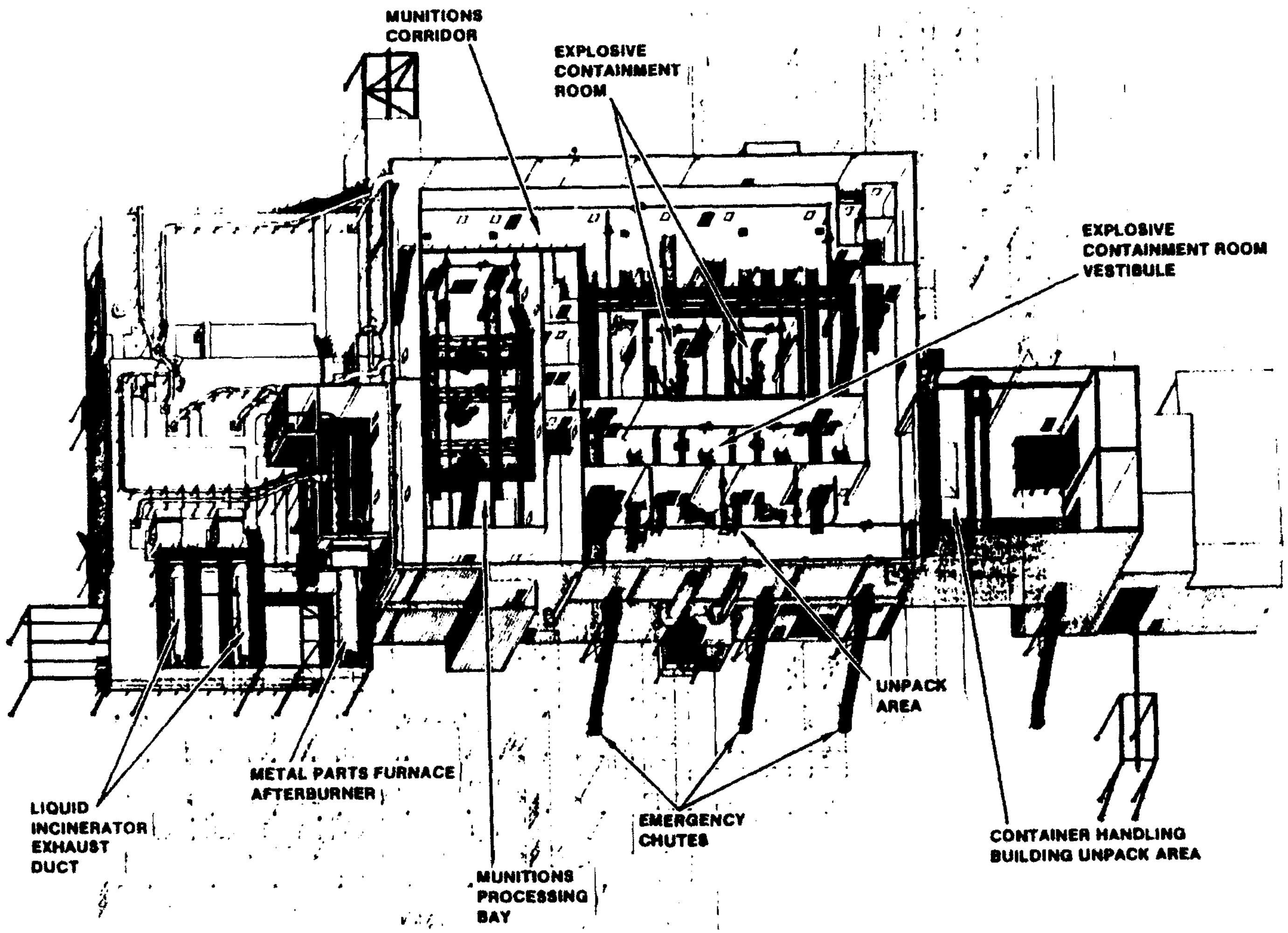


Figure 5. Munitions demilitarization building, second floor

The MDB filter area (FIL) has HVAC exhaust filters units and an exhaust stack which are located near the MDB mechanical equipment room. The filter units are designed for agent contaminated exhaust air from the MDB. The building is a structural steel frame with three monitor houses on a platform and one on slab on grade.

Nonprocess buildings

The following facilities do not house toxic chemicals and are considered standard facilities as reported in TM 5-809-10 (Headquarters, Department of the Army 1982).

The Process and Utility Building (PUB) houses bulk chemical storage, brine reduction, and boilers for steam and hot water, battery charging, and residue handling. The structural system is a braced-frame steel building with metal siding and roof panels.

Process Support Building (PSB) houses the administrative staff and is a one-story structure constructed of wood modules.

Monitor Support Building (MSB) is a one-story structure constructed of wood modules, which will house the onsite maintenance facilities for the monitoring equipment.

Entry Control Facility (ECF) is a reinforced concrete structure which houses securing personnel.

Personnel and Maintenance Building (PMB) is a single-story wood module building. The building is a support facility and includes dressing areas for employees to wear hazard protection attire.

Environmental Considerations

The life safety and environmental risks associated with the CSDP facilities are unprecedented for military construction and compare with those for nuclear power plants. To mitigate these risks and assure the safe operation and shutdown of these facilities in the event of earthquakes, highly specialized seismic design criteria were developed. The Programmatic Environmental Impact Statement (PEIS) was a major influence on the final seismic design criteria for the CSDP program. This document, prepared in compliance with the National Environmental Policy Act, assessed the health and environmental impacts of destroying the nation's stockpile of lethal chemical agents. The major focus of the PEIS was the environmental consequences of accidents involving explosions, fires, and/or spills. In support of the PEIS, a risk analysis was performed to determine scenarios which could lead to off-post fatalities. Accident initiators which were considered included processing plant equipment failures and various external events (seismic events, meteorites, tornadoes and high winds, lightning, and air crashes) and human errors. In the PEIS hazard analysis, earthquakes were a major risk contributor in plant operations, contributing about 30 percent of the total programmatic risk from onsite disposal.

With the exception of the TOX, the PEIS assumed that the facilities would be designed for seismic loads using the static lateral force procedure in the 1985 Uniform Building Code. Conservatively, the PEIS based the risk analysis for the MDB at all sites on the Tooele, UT, seismic zone (zone 3), although the order sites are in lower seismic zones. As a risk mitigative measure to minimize the potential of a release, the PEIS required that the TOX agent tanks and structural system meet more stringent seismic criteria based on Nuclear Regulatory Commission (NRC) standards.

Additional risk mitigative measures that were recommended and adopted included:

- Sizing the sump in the toxic cubicle to contain the entire contents of an agent storage tank. This reduces the spillage area, and therefore reduces the agent evaporation if the tank contents are spilled.
- Installing seismically actuated cutoff valves on the main gas supply and seismically actuated circuit breakers in the plant.

Standardized Facilities

The hazardous nature of the chemical agents being destroyed dictated that the design of the CONUS onsite disposal plants be intensely scrutinized and managed. Therefore, it was decided to standardize the designs of the onsite disposal plants. These designs would be as nearly identical as possible, given the variations in environmental conditions, siting, and inventory at each disposal site. Standardization of designs at all sites would assure the highest degree of conformance with critical safety and environmental requirements, and would mitigate the risk of all accidents including those caused by seismic events.

The design team has developed the "standard" disposal plant and will adapt it to each onsite location. This process, nicknamed "cloning," greatly reduces the design effort as well as improving configuration control and quality assurance.

Two standard disposal plant designs were developed. One design is a mixed munition plant which is capable of processing all munition configurations. The mixed munition standard design is being used at five facilities: Tooele, Anniston, Umatilla, Pueblo, and Lexington. The use of standardized facilities dictated the seismic criteria for these five sites. The Tooele site load environment was used at all five sites because Tooele is in the highest seismic zone (i.e. Zone 3) of the five sites.

The second standard is a bulk item plant. This design is to be used at locations which do not have any explosively configured munitions. This plant is required at two locations, Newport and Aberdeen.

In addition to the two standard designs, two unique designs will also be required to complete the CSDP. One design will be a CSDP plant to be located near an existing nonlethal BZ plant now in operation at Pine Bluff Arsenal (PBA). The other unique design is that of a Chemical Demilitarization Training Facility (CDTF), to be located at Edgewood Arsenal near the Program Manager's office. This facility will be used to provide highly standardized central training for all CSDP plant operators. The programmatic schedule has resulted in the first designs being the mixed munition plant at Tooele, UT, and the CDTF at Edgewood Arsenal.

Seismic Response Spectra

As required in the PEIS, the toxic room and agent tanks are to be designed to meet stringent seismic standards related to NRC criteria. The NRC criteria, specifically 10 CFR 100, require determination of the most severe earthquakes that could occur within 200 miles (320 km) of the disposal site. Therefore, studies were performed to determine the specific response spectra for the Maximum Credible Earthquake (i.e. the Safe Shutdown Earthquake) that could occur at each site (URS/John A. Blume and Associates 1987). Peak ground accelerations from this study are shown in Table 1. The response spectrum is used to design the TOX within the MDB to totally contain the stored agent in the event of the Maximum Credible Earthquake.

Table 1
Toxic Cubicle Response Spectra

Site	MCE PGA[1]	Actual Design PGA	Comment
Tooele, UT	0.81g	0.81g	Highest MCE
Umatilla, OR	0.25g	0.81g	Clone of Tooele
Anniston, AL	0.28g	0.81g	Clone of Tooele
Pueblo, CO	0.21g	0.81g	Clone of Tooele
Lexington, KY	0.18g	0.81g	Clone of Tooele
Pine Bluff, AR	0.34g	0.81g	New design
Newport, IN	0.18g	0.18g	New design
Aberdeen, MD	0.18g	0.18g	Clone of Newport

[1] Maximum Credible Earthquake; Peak Ground Acceleration.

Final Seismic Criteria

According to the PEIS, the TOX at the Tooele, UT plant was designed for the Maximum Credible Earthquake for the Tooele site.

The response spectrum method of dynamic analysis was used to analyze the TOX. The holding tank in the TOX was designed for the dynamic analysis procedures in TM 5-809-10-1 (Headquarters, Department of the Army 1986), taking into account the sloshing effect of the liquid. Piping in the TOX was not analyzed dynamically because of its small size. However, pipe supports were designed to sustain the maximum credible earthquake. The TOX at the five mixed munition sites will be standardized from the Tooele design. The TOX at the Pine Bluff and Newport will be new designs. The TOX at Aberdeen will be standardized from the Newport design. Table 1 compares the peak ground acceleration (zero period) actually used for the CSDP design with the peak ground acceleration determined by the site-specific studies discussed earlier.

With the exception of the TOX, the CSDP structures are designed for the static lateral force procedure in TM 5-809-10, which is consistent with the 1985 Uniform Building Code. All process facilities are essential and therefore have an importance factor of 1.5. All other facilities have an importance factor of 1.0. As shown in Table 2, the final CSDP criteria for seismic zones is more stringent than that required by the 1985 and 1988 Uniform Building Codes and TM 5-809-10. Based on assumptions from the PEIS and requirements due to standardization of facilities, the process facilities at all mixed munition sites are to be designed to seismic zone 3 criteria. Most of the mixed munition plant structures will be cloned from Tooele; however, some foundations at the other sites will need to be redesigned due to a lower allowable soil bearing pressure than at Tooele. In order to reduce the amount of redesign, the seismic zone requirements for the nonprocess buildings at the other sites were reduced to meet the actual site conditions, as shown in Table 2.

Table 2
CSDP Seismic Zone Comparisons

Site	UBC 85/ TM 5-809-10	UBC 88	CSDP Design Facilities: Process	CSDP Design Facilities: Non-process
Tooele, UT	3	3	3	3
Umatilla, OR	1	2B	3	2
Anniston, AL	2	1	3	2
Pueblo, CO	1	1	3	1
Lexington, KY	2	1	3	2
Pine Bluff, AR	1	1	3	2
Newport, IN	2	1	2	2
Aberdeen, MD	1	1	2	1

All equipment will be anchored to the structure in accordance with the Specification CEGS 15240, Seismic Protection of Mechanical, Electrical Equipment, using the design guidance from TM 5-809-10. This assures that the equipment remains intact after a seismic event, but not necessarily operational. With few exceptions, the equipment is designed to the same seismic zone and importance factor as the building in which it is housed. A notable exception is that the Pollution Abatement System ductwork is designed for an importance factor of 1.0 instead of the building importance factor of 1.5. The reason for this reduction is that there is no requirement for this ductwork to be undamaged after a seismic event, but there is a requirement for the ductwork to remain intact so as not to collapse and injure personnel.

Special Quality Control Criteria

In order to mitigate the risk of damage to structures, systems, or components after a seismic event, the CSDP adopted a stringent programmatic quality assurance (QA) plan. This plan encompasses the entire program, beginning with design and equipment fabrication, extending into construction and installation of equipment, and finally systemization and operations. The QA program classifies equipment, systems, and structures into three classes: (1) QA Class I items are equipment, systems, or structures whose failure or malfunction would detrimentally affect the ability to maintain containment of chemical agent or explosive effects; or influence safe shutdown or safety; or whose failure could cause an offsite release of toxic material affecting the health and

safety of the public. (2) QA Class II items are equipment, systems, or structures which could cause shutdown of the process. (3) QA Class III are items of equipment, systems, or structures which cannot adversely effect the process or safety requirements of the program. These items are procured to normal industry standards, codes, and constructed to normal specifications.

As a result of this QA classification system, equipment and structures which if damaged during a seismic event that could cause release of agent, loss of life, or a process shutdown require an inordinate amount of inspections, certifications, and factory testing. As one example, as was previously mentioned, the TOX was identified as a major source of agent release in the event of an earthquake. As a result, this structure is classified as a QA Class I structure, which requires stringent quality assurance requirements such as (1) certified mill test reports for materials of construction, (2) greater number of sampling and testing than required normally, (3) placement plans, (4) strict identification of contractor inspection hold points, (5) weld testing in the form of radiography and ultrasonics for 100 percent of all full penetration welds, (6) random testing of all other welds, and factory performance tests for equipment within the structure and for components such as floor sumps.

As is apparent, this QA program is very costly and adversely affects construction productivity. Critical equipment, systems, and structures which could be damaged during a seismic event have been carefully identified to assure safe operations. For these items, stringent fabrication, testing, construction, and installation documentation and testing have been programmatically defined, and have been given the highest priority in assuring a safe and operational facility.

Lessons Learned

A major lesson learned from the CSDP is that environmental permit considerations can dictate the overall facility design, including the seismic design criteria. The PEIS assumed more stringent seismic design criteria than required by TM 5-809-10. The CSDP design was required to incorporate these stricter requirements. If the PEIS requirements had not been incorporated, the CSDP design and construction schedule would have been severely disrupted due to additional environmental reviews.

Numerous design problems have been encountered due to the stringent seismic design requirements of the Tooele, UT facility. For example, several equipment vendors did not have a clear understanding of the special seismic design requirements for equipment anchorage in TM 5-809-10, which are more extensive than those in the Uniform Building Code. Some vendors did not correctly follow the procedures in TM 5-809-10 which are controlled by the equipment weight and flexibility. Therefore, several equipment shop drawings and calculations had to be revised and resubmitted for review, causing delays in the programmatic schedule.

The seismic design requirements have created numerous construction difficulties at the Tooele, UT CSDP facility. A major problem is the tightly spaced reinforcement in the concrete shear walls in the Munitions Demilitarization Building. These shear walls have numerous penetrations for mechanical and electrical equipment. In order to meet ACI special seismic design requirements, the stirrup spacing cannot be greater than one-quarter of the wall thickness. Most of the shear walls are 10 in. thick with a required stirrup spacing around the penetrations of only 2-1/2 in. Since the other four mixed munition plants are clones of the Tooele facility, the same shear wall reinforcement and spacing will be used at those sites. From a construction standpoint, the special seismic design requirements have caused many difficulties and are costly to the overall program.

Despite the design and construction problems, the special seismic design criteria are necessary to meet critical safety and environmental concerns in the event of an earthquake. The special design criteria have satisfied the guidelines of the CSDP which

are to assure safe operations during the chemical stockpile disposal process.

References

Headquarters, Department of the Army. 1988 (Jan). "Chemical Stockpile Disposal Program Final Programmatic Environmental Impact Statement." Program Executive Officer-Program Manager for Chemical Demilitarization, Aberdeen Proving Ground, MD.

Headquarters, Department of the Army. 1982 (Feb). TM 5-809-10, "Seismic Design for Buildings," Washington, DC.

Headquarters, Department of the Army. 1986 (Feb). TM 5-809-10-1, "Seismic Design Guidelines for Essential Buildings," Washington, DC.

URS/John A. Blume and Associates Engineers. 1987 (Oct). "Geological - Seismological Investigation of Earthquake Hazards at Sites for Chemical Agent Demilitarization Facilities."

Steel Deck Diaphragm Design Methods: Tri-Services Manual vs. Steel Deck Institute

by
Chris Glatt, PE[1]

Abstract

For Corps of Engineers projects, the design of shear diaphragms constructed from light-gage corrugated metal deck is usually governed by the Tri-Services Manual 5-809-10 (US Government Printing Office 1982). The Diaphragm Design Manual (Steel Deck Institute 1981, 1987) is also used, mainly for diaphragms with nonwelded connections. In this paper, the two methods are briefly described, and their results are compared to determine if they predict equivalent design shear strengths. Design strength values are compared for diaphragms most commonly used on military building projects in the US Army Engineer District, Kansas City. In addition, results from several sensitivity studies are compared to determine the influence of individual parameters. These results indicate that the methods do not always agree, and additional research is needed to evaluate areas of disagreement between the methods.

Introduction

Background

Shear diaphragms used on Corps of Engineers building projects are commonly constructed from light-gage, corrugated metal panels, which are connected at the panel sidelaps ("seams") to interior structural supports (purlins, joists, bearing walls, etc.) and to edge and end members (Figure 1). These connections are at discrete points and may be made with welds, screws, power-driven pins, or a combination thereof. Under shear loading, these diaphragms undergo shear deformation which is affected by warping of the corrugations and by localized deformations at the fastener locations. This complex behavior has been the subject of much testing and analysis, leading to the development of several design specifications, including the two discussed in the following paragraph.

The design and construction of horizontal diaphragms for most US military facilities is governed by the Tri-Services Technical Manual 5-809-10 (US Government Printing Office 1982), hereafter referred to as the TM. Another commonly used design method is that of the Steel Deck Institute (SDI), the *Diaphragm Design Manual* (SDI 1987). The SDI criteria are being applied with increasing frequency on recent US Army Engineer District, Kansas City (KCD), military building projects for two main reasons. First, more steel decks are being installed with nonwelded fasteners such as screws or power-driven pins. In this case, the SDI criteria are used because the TM does not address nonwelded diaphragms except to say "Fastening methods other than welds...may

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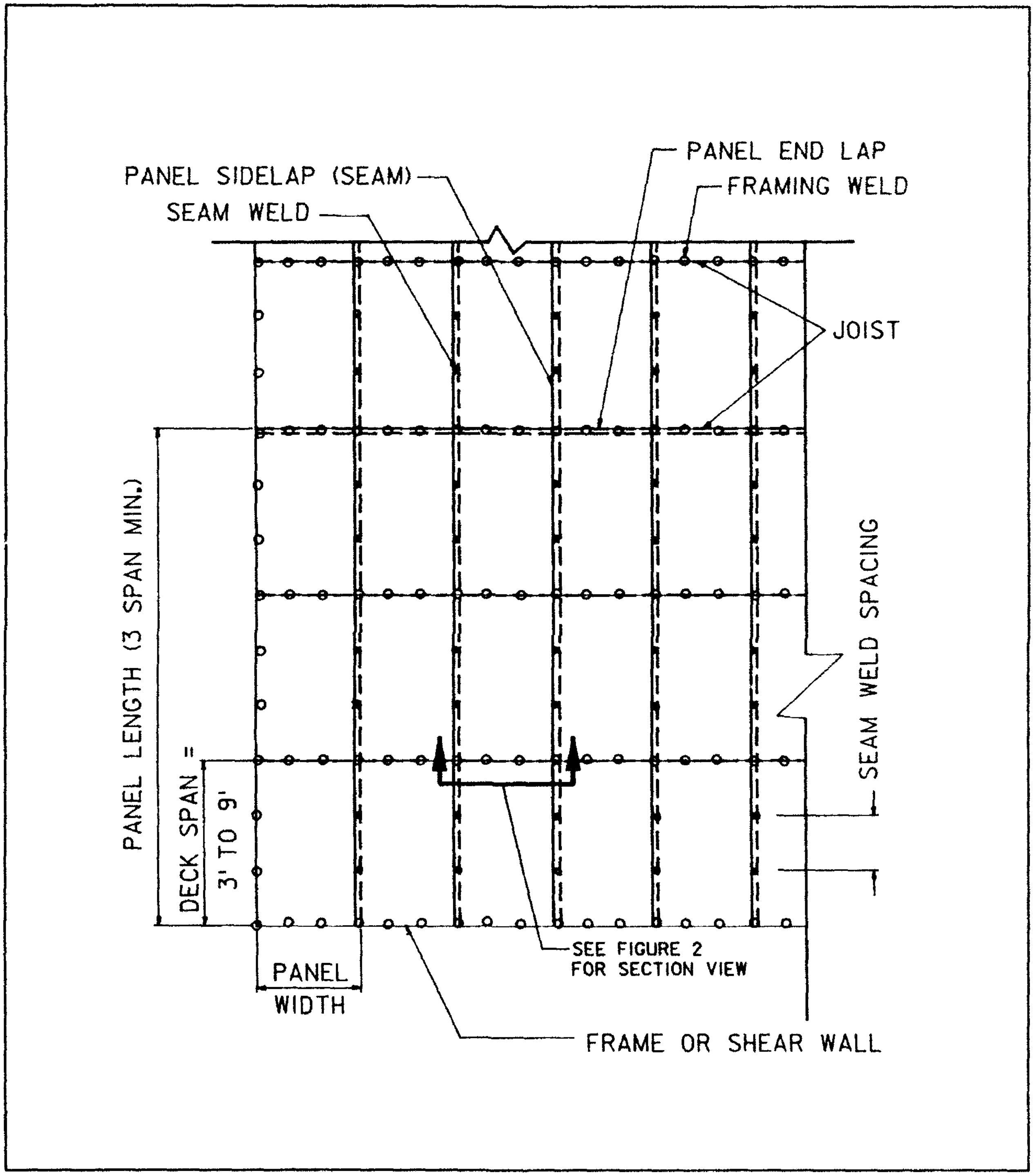

Figure 1. Typical diaphragm configuration, plan view

be used provided that equivalence to the welded method can be shown by approved test data. The results of such test data will be presented by means of equations or tables for q_D and F..."(q_D and F are the diaphragm design shear strength and flexibility). Since the SDI equations are based on testing and their results can easily be expressed in terms of q_D and F, the TM requirement can be interpreted as allowing the use of SDI design criteria. The other reason SDI criteria are used is that more projects are being designed to meet industry or commercial standards; in other words, no "military" criteria are allowed if industry standards (such as SDI criteria) exist. Either of these methods can be used for diaphragm design without cause for concern, as long as they give the same, correct answers; i.e., they predict the actual diaphragm behavior with reasonable accuracy. With this in mind, the most common types of diaphragms used on KCD projects were studied. The following paragraphs include a brief discussion of the two methods, values for diaphragm strengths calculated using both methods, evaluations of these results, conclusions, and recommendations for further studies.

Scope of study

This paper is restricted to consideration of diaphragm shear strengths for diaphragms with all welded connections. Although stiffness is an equally important design consideration, it is discussed only briefly, with emphasis on its relationship to strength calculations. It is also worth noting that the number of diaphragms studied was small, considering the wide variety of deck panel profiles, thicknesses, widths, and fastener patterns available.

The approach taken was to define a "typical" diaphragm, as follows, and compute design shear strengths using both methods. Then sensitivity studies were made to assess the influence of several variables.

The typical diaphragm was defined by considering common loading requirements for KCD projects. Since design loads due to earthquake or wind are relatively moderate in the Midwest, designs for regularly shaped low-rise buildings usually require diaphragm design strengths of about 200 to 400 pounds/lineal foot (plf), and stiffnesses of about 10 kips/in. Also, the most economical framing system for vertical loads typically results in deck spans of 4 to 6 ft between open-web steel joists. These two considerations usually lead to the use of a 22-gage, 1-1/2-in.-deep, single-sheet deck with relatively few welds. Therefore, for comparison purposes in this paper, the typical diaphragm was defined as a 22-gage, narrow rib deck welded to the framing in a 36/4 pattern (the first number is the panel width, the second the number of framing welds across the panel), with two 1-1/2-in. seam welds per span. Using the typical diaphragm results as a baseline, several studies examined the effect of changing one variable, over a range of values commonly found on KCD projects. These sensitivity studies addressed the following parameters (values studied are shown in parentheses and illustrated in Figure 2):

- Deck vertical load span (3 to 9 ft).
- Deck thickness (22, 20, and 18 gage).
- Deck type (narrow and wide rib).
- Framing weld pattern (36/4, 36/7, 30/6, and 30/3).

It is emphasized that a detailed analysis of how or why the methods work (or do not work) is not within the scope of this paper; instead, the aim is to see if the results of the methods agree and to identify the types of diaphragms for which the results do not agree.

Description of Methods

SDI method

The SDI approach to steel deck diaphragm design is based on theory as well as testing. Theoretical strength formulas have been developed, based on static equilibrium, for three fastener failure modes. Another strength formula has been developed based on an overall buckling limit state. Similarly, a general

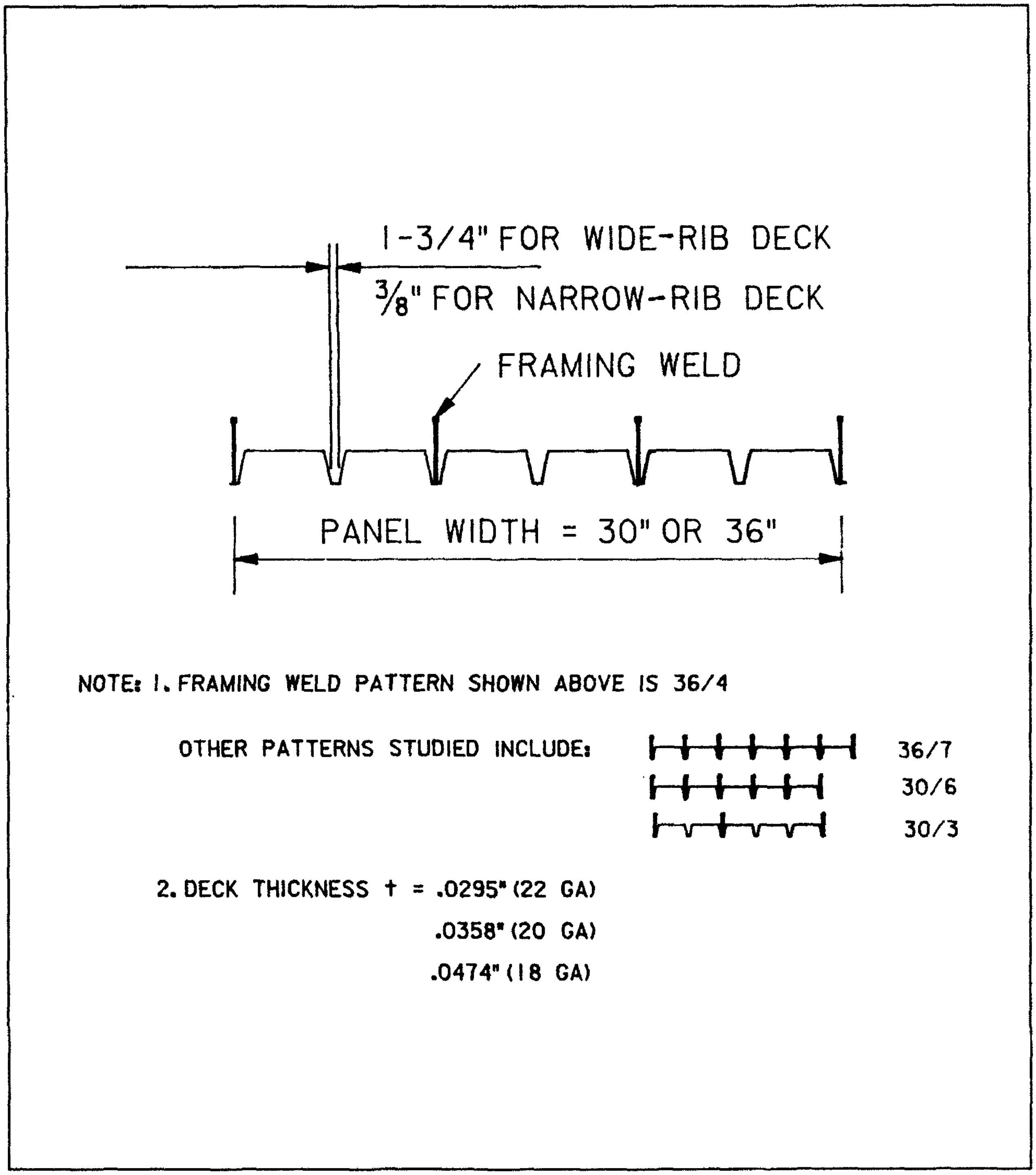

Figure 2. Typical diaphragm configuration, section view

stiffness equation has been developed based on theory of elasticity and modified to account for panel warping and connector slip. A brief description of the equation development follows; for detailed discussion of the equations and definition of the terms therein, the reader is referred to the SDI manual (SDI 1981 and 1987).

The SDI strength and stiffness equations all are generic with respect to the fastener type, so that the strength and stiffness of a particular diaphragm can be evaluated as a function of fastener type. In other words, for a given diaphragm system, a change in connector type will affect the diaphragm response only to the extent that the connector response changes. These formulas have been verified by large-scale tests of diaphragm systems. In addition, equations have been developed to predict the strengths and stiffnesses of common fasteners such as welds, screws, and power driven steel pins, based on individual fastener tests. The difference in reliability for the different fastener types is accounted for in the safety factors used to relate the ultimate diaphragm strength to the design strength. These safety factors were developed using "Load and Resistance Factor Design" (LRFD) methods which utilize statistical techniques to reflect the variations in connector quality and the likelihood of potential overloads. The safety factors recommended by SDI for welded and mechanically fastened diaphragms are 2.75 and 2.35, respectively.

The results of these strength and stiffness equations have been tabulated by SDI, for diaphragms fastened with welds, screws, and power-driven steel pins. The tabulated values were developed using the following assumptions:

- Steel $F_y = 33$ ksi, $F_u = 45$ ksi.
- Deck profiles match the typical profiles given in SDI Publication No. 27 (SDI 1989).
- Deck panels span four supports (e.g., joists)
- The number of diaphragm edge connectors along the deck span is equal to the number of sidelap connectors along the deck span.
- The framing weld pattern at interior supports is the same as at panel ends.

TM method

The TM design approach is also based on theory as well as full-scale diaphragm tests. The TM equations address only two types of diaphragm connectors - welds and button-punches (button-punches are used only for sidelap connections). The TM does not include background information explaining how the equations were developed, nor are there any published documents containing such information; however, the researchers who developed the equations have documented the work in an unpublished report, which they graciously provided to the writer. A brief summary of that report follows.

The general TM equation for diaphragm shear strength contains three terms, each of which includes coefficients calibrated by full-scale testing so that the predicted overall diaphragm strength provides a safety factor of 3.0. An additional strength equation (Equation 5-10) addresses local buckling of the panel edge flute between sidelap connections. This equation applies only when the edge flute width is less than 1/2 in., which is true for narrow-rib decks. In fact, the edge flute width for intermediate rib decks is only 5/8 in., according to the SDI profiles given in Reference 4 of the TM. Therefore, it is questionable if this equation should apply for intermediate rib decks as well.

The results presented in the following pragraphs were developed using the same assumptions as described in section 2.1 of the TM for the SDI tabulated values, except the TM equations are based on $F_u = 55$ ksi instead of 45 ksi.

Differences between methods

One general difference between the SDI and TM methods is that the SDI method isolates fastener properties, so that their effect on diaphragm properties can be evaluated relatively quickly. Another general difference is that in the TM equations, strength and stiffness terms are interdependent, while the SDI equations address them separately. Consequently, the SDI method makes it easier to isolate the effects of the individual parameters on strength or stiffness.

In addition to these general differences, the following specific differences in the SDI and TM formulations exist. First, the type of deck or corrugation profile is accounted for in both strength and stiffness calculations for the TM (by means of the moment of inertia terms), while in the SDI equations, deck type is not a factor in diaphragm strength. Second, the size of framing welds is not a variable in the TM equations as it is in the SDI equations (the results presented below used 3/4-in.-diam welds for the TM values and 5/8-in.-diam welds for the SDI values). Third, the sheet steel yield or ultimate strengths are not variables in the TM formulas as they are according to SDI (the values presented below are based on $F_u = 55$ and 45 ksi for the TM and SDI, respectively). However, according to the SDI manual, "In round welds made without washers, the material strength F_u may not have great significance especially when it is below 60 ksi." Finally, the factor of safety used in the TM strength equations is 3.0, or about 10 percent greater than the SDI safety factor of 2.75.

In addition to these differences in the **formulations** used by each method, the results may differ due to factors affecting the actual test data used in developing each method. One such factor is the amount of data generated. According to persons with knowledge of the testing done for these methods, the SDI method is based on several hundred tests performed within the last 15 years, while the TM testing consisted of less than 100 tests done in the 1960's. It stands to reason that the repeatability or reliability of the TM test data would be more suspect. The reliability of the weld strengths predicted by the SDI equations has been thoroughly studied, through LRFD analyses based on many small-scale tests; in comparison, the reliability of the TM weld strengths is relatively unknown.

Another factor which could skew the test data is the quality of welding used in the test assemblies. The welding done for the SDI tests was reportedly intended to represent realistic field-type welding. The test assemblies used for the TM tests were reportedly welded by highly skilled welders, for corporate entities (deck manufacturers) interested in obtaining the maximum performance from their product. It would be difficult to establish if, in fact, the welding quality differed for the two testing programs; nonetheless, it should be considered as a valid explanation for disagreement between the results of the two methods. It is critical that welding done for diaphragm tests represent field-quality welding, since the performance of the finished diaphragm is highly dependent on the skill of the welder. In fact, the SDI manual recommends against using sidelap welds for sheets 0.0295 in. (22-gage) or less in thickness, due to the inherent difficulty of making such welds. This concern about field welding quality is especially significant for KCD projects, because the contract is usually awarded to the low bidder, who then performs his own quality control. Recent experience on KCD projects has shown that contractor quality control has contributed to cases where construction quality did not meet designers' expectations.

Analytical Method

Results were analyzed by plotting values for design shear strength from the TM and SDI equations on the same graph, for varying span lengths. The data from SDI equations were taken directly from tables provided in the SDI Manual. Results from TM equations were generated using a short FORTRAN program, with some input variables calculated by hand. Program output was checked with hand computations and output of other programs.

Results and Evaluation

Typical diaphragm

Strength values for the typical diaphragm (22-gage, narrow-rib deck, 36/4 framing weld pattern, 2-seam weld/span) predicted by the SDI and TM equations are shown in Figure 3a. The two curves are essentially parallel, with strengths increasing almost linearly as span lengths decrease. The SDI values consistently exceed the TM values by about 65 plf, or about 20 percent of the average SDI strength. About one-half of this difference is attributed to the higher safety factor used in the TM equations (3.0 compared to 2.75 for the SDI equations).

Sensitivity studies

The sensitivity of the diaphragm strength to deck thickness, for 18- and 20-gage decks, is also shown in Figure 3a. Note that, although the shape and slope of the curves are similar, the TM values increase substantially more than the SDI values, as deck thickness increases from 22 to 18 gage. As a result, the two methods agree within 10 percent for the 20-gage deck, and the TM values for the 18-gage deck range from 10 to 30 percent greater than the SDI values.

The second sensitivity study considered the deck profile. The SDI strengths shown in Figure 3a apply to all deck types. The TM values, however, increase for wide-rib decks compared to narrow rib decks (see Figure 3b). This difference is attributed almost entirely to TM Equation 5-10, which is based on local buckling of the panel edge flute. Although the SDI equations address the same behavior ("strut-like buckling," in SDI terms), the SDI equations do not include deck type as a separate variable. Note that for 22-gage decks, the influence of Equation 5-10 is significant only for span lengths less than 4 ft. Thicker decks, however, are increasingly sensitive to this equation.

The effects of the framing weld pattern, the third variable studied, are shown in Figures 4a and b. Results shown in Figure 4a show how SDI strengths vary for 36/7, 30/6, 36/4, and 30/3 patterns. A corresponding set of curves for the TM data is shown in Figure 4b. For 22-gage decks, the change in diaphragm strengths follows the same trend for all weld patterns, according to both methods, with the strength slightly more sensitive to the weld pattern as spans decrease.

Stability checks

Overall buckling of the diaphragm is another failure mode which was considered. Both methods include provisions limiting the design strength based on the critical buckling load. Results from these equations show that buckling does not control over the other strength equations unless deck spans are well over 10 ft and many fasteners are used. Since such diaphragms are seldom used in buildings, more detailed studies were not made.

Summary and Conclusions

The most obvious conclusion indicated by these results is that diaphragm behavior is complex, due to the number and interaction of variables involved. It is difficult to make general statements which apply to all of the diaphragms studied. It is overemphasis to repeat that the main purpose of this report is to identify areas where the TM and SDI methods disagree, to pinpoint **why** the predicted results differ, or which results more accurately reflect actual diaphragm behavior. With that in mind, the following conclusions were drawn, based on the results presented.

- For the typical diaphragm (22-gage, narrow-rib panels fastened with few welds), the TM equations predict slightly lower strengths than the SDI equations.

- The results of the SDI and TM equations are equally sensitive to two of the variables studied, and support observations reported by others (SDI 1987). These variables, and their effect on diaphragm response, are as follows:

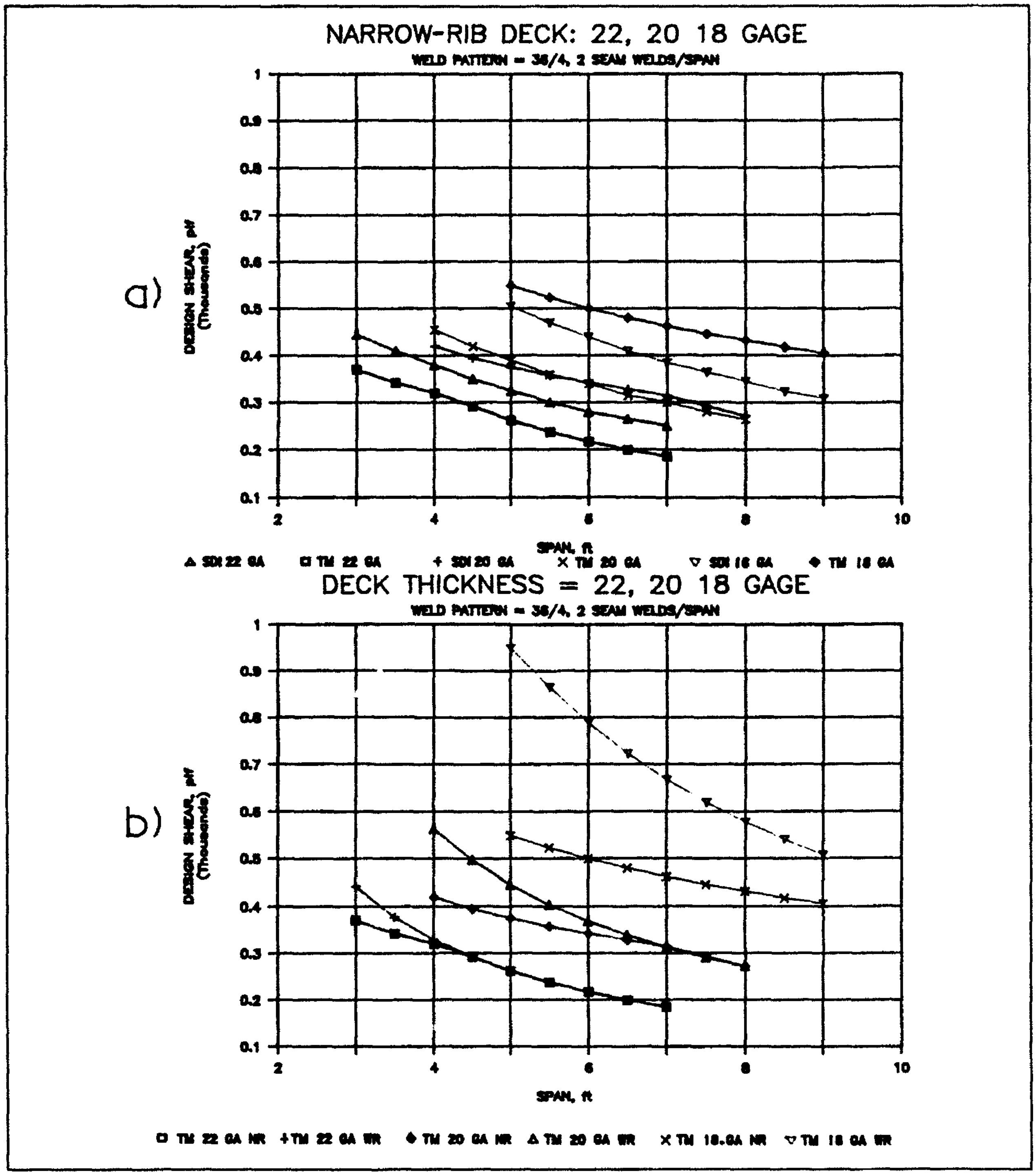

Figure 3. Deck thickness and type sensitivity

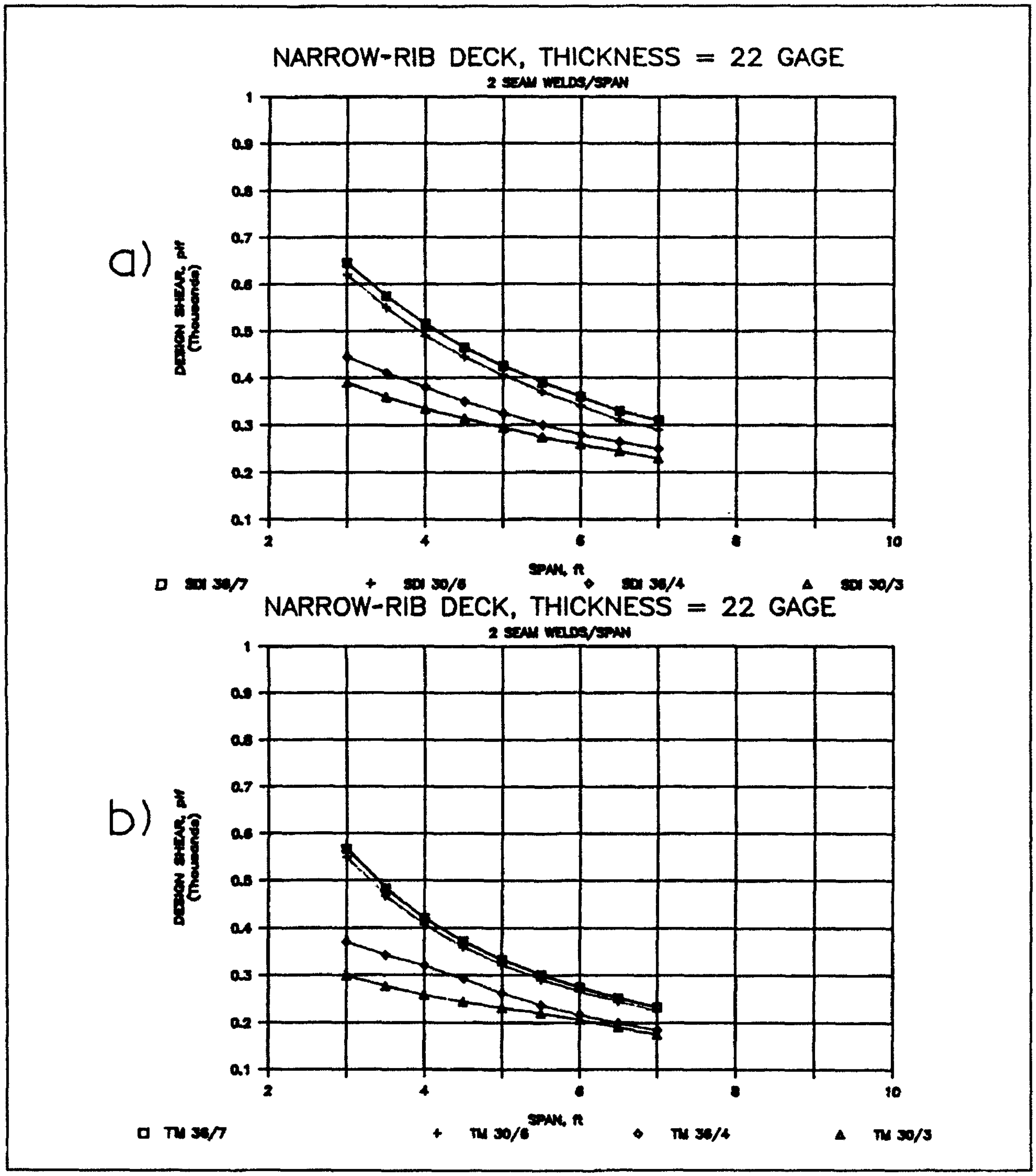

Figure 4. Framing weld sensititivy

* Span - Strength increases almost linearly, with decreasing span length.

* Framing weld pattern - Strength increases as the spacing of framing welds is reduced.

- For the remaining variables studied, the results of the SDI and TM equations are not in good agreement. These parameters and their predicted effects on diaphragm response are:

  * Deck thickness - Strength increases almost linearly with increasing thickness according to both methods, but the TM equations are much more sensitive to deck thickness.

  * Deck type - According to the TM equations, wide-rib decks are stronger than narrow-rib decks, especially for shorter spans and thicker decks. The strengths predicted by the SDI equations are not affected by deck type.

- Assuming that the TM equations were correctly applied when developing data for these comparisons, the reasons the SDI and TM results differ could include the following:

  * The data from the testing done to develop both methods agree, but the equations do not accurately fit the data, or the equations extrapolate beyond the range of test data. This seems unlikely given the amount of time and effort expended in developing each method.

  * The equations fit the data, but the data disagree due to differences in the methods or models used in the testing programs. Such differences include the following:

    ¤ Framing weld diameter (3/4 in. for TM, 5/8 in. for SDI).

    ¤ Steel strength ($F_u = 55$ ksi for TM, 45 ksi for SDI).

    ¤ Welding quality (see previous paragraph).

Obviously, more detailed knowledge of full-scale diaphragm tests is needed to assess which method is "correct" (i.e., more representative of most actual installations). **In the writer's opinion**, the first two factors cited above are plausible reasons why the TM strengths would tend to increase more than the SDI strengths as deck thickness increases. However, they may be minor compared to the third item - the welding quality. For the reasons discussed above, the quality of welding not only emphasizes the effects of the two variables just described, but is also a critical concern because the value of the TM design method is questionable if it is based on laboratory conditions that do not represent actual construction practices.

No specific reasons for the other major area of disagreement - the effect of deck type or corrugation profile (rib width) on diaphragm strength - were found during this investigation.

Recommendations for Further Study

Based on this above discussion, it is recommended that more detailed evaluations of the two methods be made to determine which more accurately predicts actual behavior of steel deck diaphragms as constructed in the field with emphasis on decks of at least 18-gage thickness. The type of deck should also be investigated to assess its influence on diaphragm response. In particular, attention should be given to the quality of the welded connections to determine if the tested diaphragms are representative of realistic construction practices.

References

Steel Deck Institute. 1981. *Diaphragm Design Manual*, 1st Edition, Canton, OH.

Steel Deck Institute. 1987. *Diaphragm Design Manual*, 2nd Edition, Canton, OH, 1987.

________ . 1989. *Design Manual for Composite Decks, Form Decks, Roof Decks, and Cellular Metal Floor Deck with Electrical Distribution*, Canton, OH.

US Government Printing Office. 1982. *Seismic Design for Buildings*, Tri-Services Technical Manual TM 5-809-10; NAVFAC P-355; AFM 88-3, Chapter 13, Philadelphia, PA.

Seismic Structural Engineering Research at the Corps of Engineers Laboratories

by

Dr. Robert L. Hall[1] *and John R. Hayes, Jr., PE*[2]

Abstract

This presentation provides a brief overview of the combined research capabilities, ongoing research programs, and planned research activities at the US Army Construction Engineering Research Laboratory (CECER) and the US Army Engineer Waterways Experiment Station (CEWES) in the seismic structural engineering arena. The presentation serves as basic familiarization for those who are unfamiliar with the seismic research activities at the two laboratories.

Introduction

The US Army Corps of Engineers (USACE) constructs and maintains infrastructure in seismically active regions both in the United States (US) and internationally. In the US, both the large USACE Civil Works programs and the military construction programs for the Army and Air Force include work in high seismic risk zones; internationally, USACE military construction programs extend to high seismic risk regions. The military facilities are likewise maintained by Engineering an Housing personnel under the general guidance of the Engineering and Housing Support Center (CEHSC). USACE has also become involved in postearthquake disaster recovery efforts in support of the Federal Emergency Management Agency.

Because of this extensive USACE involvement in structural engineering in areas potentially affected by large magnitude earthquakes, CEWES and CECER have developed seismic engineering capabilities to perform both long-term research on USACE-unique problems and to assist USACE agencies, principally CECW, CEMP, and the various division and district offices with specific technical problems that arise. This paper concentrates on the seismic structural engineering research and capabilities that exist at the two laboratories, but it is important to note at the outset that a key component of research, design, and analysis in the seismic field is geotechnical engineering. At CEWES, the Earthquake Engineering and Geosciences Division of the Geotechnical Laboratory (CEWES-GG) is responsible for seismic research and analysis to support all USACE mission areas, both military and civil works. CEWES-GG works in the research arena with structural engineers at both CEWES and CECER and also provides needed analysis to support field activities.

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The structural engineering research activities at CECER are conducted by the Structural Engineering and Physical Security Team of the Engineering and Materials Division (CECER-EM-E); at CEWES, they are conducted by Structural Analysis Group of the Structural Mechanics Division, Structures Laboratory (CEWES-SS-A). In general, the two groups have separate, but complementary, research thrusts driven by historical organizational mission areas. At CEWES-SS-A, research concentrates on technical issues associated with the construction of large-mass civil works structures (e.g. locks and dams); these structures pose unique foundation, structural, and fluid-structure interaction phenomena that are largely unique to the USACE Civil Works infrastructure, traditionally a primary CEWES research support area. The structural work is complemented by the geotechnical and fluid-structure interaction capabilities found in other divisions at CEWES. On the other hand, CECER-EM-E performs research related to building structures, consistent with the traditional CECER research to support construction on military installations. This work focuses on the more flexible frame and shear wall building construction found at Army posts and Air Force bases. CECER structural research is complemented by the architectural, planning, and other talents there that are required for successful facility design research.

Both CEWES and CECER perform research under the auspices of USACE-funded research, development, test, and evaluation (RDT&E) programs controlled by the Directorate of Research and Development (CERD) and supported by CECW and CEMP. These programs are devised to foster long-term research focus in USACE problem areas. In addition, both laboratories have performed, and continue to perform, numerous research projects for USACE and other clients on a cost-reimbursable basis; such projects tend to have more short-term focus that support the customers in solving immediate problems.

This paper briefly describes the seismic structural engineering research capabilities at CECER and CEWES, their recent past and ongoing seismic research initiatives and their planned future research activities. USACE, and indeed all federal agencies, have new impetus to maintain these capabilities and promote research on both building structures and civil works facilities. On 16 November 1990, Congress passed Public Law 101-614, the National Earthquake Hazards Reduction Program (NEHRP) Reauthorization Act. The act requires, among other items, that "The President shall adopt, not later than December 1, 1994, standards for assessing and enhancing the seismic safety of existing buildings constructed for or leased by the Federal Government which were designed without adequate seismic design and construction standards." Beyond the obvious concern that engineers should have for life safety, USACE will be involved with national efforts to comply with this law. The laboratories can serve a vital role in these efforts.

Seismic Structural Engineering Research at CEWES

CEWES seismic research has focused on employing improved and more rigorous analytical techniques to be used in designing large lock and dam structures. Such techniques, and the needed geotechnical support, are not typically available to division or district personnel. Key areas of complexity found by CEWES include soil-fluid-structure interaction, temporal-spatial loading phenomena associated with long-period seismic waves, and system response phenomena.

CEWES has a broad capability to perform both analytical and experimental structural engineering research. For analytical research, CEWES has a variety of linear and nonlinear finite element programs available and has both minicomputer (Silicon Graphics workstations and MicroVax II) and supercomputer (Cray Y-MP) capabilities. A full contingent of field test personnel and instrumentation is complemented by newly acquired forced vibration testing equipment, including 1,000 and 2,000-lb force electrohydraulic inertial mass shakers (two each), and three 50-lb

force electrohydraulic shakers. This vibration testing equipment is further complemented by a ruggedized laptop 80386-based microcomputer for data acquisition and analysis and identification of linear and nonlinear system dynamic response parameters. Within the CEWES Structural Mechanics Division, there are 38 engineers with 68 degrees from 22 different universities. This large, well-qualified in-house work force is augmented by seismic experts form prominent universities and private engineering firms. Professors from the University of California at Berkeley, North Carolina State University, West Virginia University, and Oklahoma State University have recently performed various tasks for CEWES under contract. Finally, the CEWES Graduate Institute affords CEWES the opportunity to utilize professors at Mississippi State, Louisiana State, and Texas Tech universities.

Key Research Projects

Analysis of the Seven Oaks Dam

This project was performed for the US Army Engineer District, Portland. The proposed dam is a major flood control project located approximately 2 km upstream from the mouth of the Santa Ana River Canyon, near the south branch of the San Andreas Fault. The north branch of the fault crosses the reservoir about 1 km upstream from the proposed dam site. The analysis emphasized the outlet structures associated with the dam, including the intake tower, which leans against an adjacent abutment, and the gate chamber for the dam, which is embedded in rock and connected to the tower by a pressurized tunnel. The site's projected Maximum Credible Earthquake (MCE) had a peak ground acceleration (PGA) of 0.7 g; the Maximum Probable Earthquake (MPE) PGA was 0.5 g. Three-dimensional finite element analyses were performed on the structures; the analyses concentrated on potential separation of the tower and the abutment, and associated pounding during a major earthquake. Response spectrum analyses were performed on the gate chamber; response spectrum analyses and time-history analyses were performed on the intake tower. Unique features of the analyses included three-dimensional ground motion input, added mass to account for reservoir effects, and inclusion of rock abutment flexibility.

Analysis of the Folsum Concrete Gravity Dam

This project was performed for the US Army Engineer District, Sacramento. The dam is located on the American River, about 20 miles upstream from the city of Sacramento, CA. The seismic safety of the structure was evaluated for a magnitude 6.5 earthquake's occurring on the East Branch of the Bear Mountains Fault Zone; anticipated PGA was 0.35 g. The analysis focused on the concrete gravity dam section of the overall project. It used a state-of-the-art twodimensional finite element analysis technique developed by the University of California at Berkeley, EGAD-84, to determine the maximum principal tensile stresses in the outer faces of the dam. Based on these analyses, CEWES concluded that the dam will maintain its integrity during and after the anticipated earthquake.

General analysis of overturning stability of intake towers

Since the intake tower is a key component of any dam system, and its relatively high center of gravity provides a large potential for overturning, CEWES undertook a study to develop simplified rational methods of determining overturning potential using the spectral accelerations and velocities associated with first mode responses of intake towers to seismic ground motions.

Simplified seismic analysis of concrete gravity dams, including foundation stiffness

In research for CEWES by North Carolina State University, a two-dimensional finite element model, SDFDAM, was developed. The model uses the key observations that horizontal ground motions and first-mode structural responses are the most significant parameters in dam load-response analyses to simplify

calculational procedures. Structural responses are modified to account for rock-foundation and structure-water interactions.

Olmsted Lock preliminary dynamic analysis

This project was performed for the US Army Engineer District, Louisville. CEWES used the commercial nonlinear dynamic finite element analysis program ADINA to perform this analysis. The analysis was again a two-dimensional approach. The unique aspect of this project was that, unlike any other civil works projects in areas of potentially high seismicity, the proposed structure was founded on piles, which had to be simulated in the analysis.

Seismic vulnerability analyses of strategic air command bases in California. Headquarters, Strategic Air Command (HQ SAC) supported this project. The work was performed jointly by CECER and CEWES. This project consisted of preliminary seismic vulnerability screening and analyses of the facilities at Beale, Castle, and March Air Force Bases. CEWES-GG performed geotechnical evaluations to determine anticipated PGA for the analyses. CEWES-SS-A and CECER-EM-E jointly performed site visits to perform preliminary screenings. CECER-EM-E led the structural analysis work, assisted by CEWES-SS-A.

CEWES-SS-A has developed a comprehensive long-range seismic research program that concentrates upon improved design and analysis procedures for Civil Works structures and has begun developing closer ties to other federal agencies involved in seismic research, particularly the National Institute of Standards and Technology (NIST) and the National Science Foundation. Key proposed research projects include:

- **Seismic response of locks.** This program would develop formalized procedures for seismic analyses of locks. New guidance would cover selecting optimum analysis methods, applying external loads, and determining whether two-dimensional or three-dimensional models are required. Added computational tools would also be developed. Proposed for FY92-FY96.

- **Seismic response of reinforced concrete structures.** This project would develop comprehensive design guidance for Civil Works reinforced concrete structures. Special attentions would be paid to reinforcement details, ductility requirements, and shear strength. Proposed for FY92-FY95.

- **Seismic response of dams.** This project concentrates upon bringing state-of-the-art research results together in a design guidance document. Topics to be covered include response spectra criteria, effects of traveling seismic waves on long structures, effects of reservoir bottom absorption, stability, nonlinear structural effects, effects of vertical ground motion components, and calculation of traditional moment, shear, and thrust values from finite element calculations.

- **Seismic response of outlet works.** This project will develop design and analysis procedures for intake-outlet structures. The project will key on recent research results at the University of California at Berkeley, in which water mass effects on intake towers were studied. In addition, the interactions of intake towers with their access bridges will be analyzed. Proposed for FY93-FY97.

- **Seismic structural risk analysis**. This project will apply structural engineering reliability analysis procedures to Civil Works structural design and evaluation. The vehicle for initiating the project will be a workshop of reliability analysis experts. Ultimately, a microcomputer-based procedure will be developed to assist engineers. Proposed for FY93-FY97.

- **Urban search and rescue.** This project will develop guidance for employing expedient measures to aid in search and rescue for victims trapped in structures after

natural disasters occur. The guidance will cover rapid shoring, excavating, and cutting techniques. Proposed for FY92-FY96.

Seismic Structural Engineering Research at CECER

Seismic structural engineering research at CECER has emphasized building structures. The majority of CECER's seismic research since 1970 has centered upon supporting design guidance that is disseminated through the triservice technical manuals, TM 5-809-10, "Seismic Design for Buildings"; TM 5-809-10-1, "Seismic Design Guidelines for Essential Buildings"; and TM 5-809-10-2, "Seismic Design Guidelines for Upgrading Existing Buildings" (Headquarters, Department of the Army, 1982, 1986, and 1988, respectively). From FY72 through FY88, CECER produced a series of approximately 20 technical reports on seismic design of building structures, most of which led directly to procedures outlined in the triservice manuals.

A unique combination of factors has influenced the evolution of the CECER seismic research program. First, the nearby University of Illinois at Urbana-Champaign (UIUC), with its renowned structural engineering research capabilities, has been a solid seismic research partner; UIUC professors have worked under contractual arrangements, as informal mentors and as cooperative research partners with CECER-EM-E. In addition, UIUC graduate students frequently work as graduate assistants to the eight full-time CECER-EM-E structural researchers, providing valuable assistance and insights. The UIUC laboratory, computational, and library facilities are also available to CECER staff members; and, CECER engineers can take graduate courses on campus. The second unique characteristic at CECER is the presence of the Biaxial Shock Test Machine (BSTM), one of this country's three large public sector shaking tables. The BSTM is a valuable tool that can be used to simulate earthquake ground motion effects on model structures and structural components. The BSTM is complemented by a large structural load floor and associated testing equipment, including several load actuators and a 1,000,000-lb MTS load frame. CECER has balanced these academic and experimental capabilities with contract access to prominent US engineering firms. Finally, CECER-EM-E is complemented by the other building research talents within CECER, especially the architectural and construction engineering talents that link structural engineering research to the comprehensive design, construction, and maintenance processes associated with military construction.

With the obvious emphasis that is being placed upon evaluating and upgrading existing facilities, the CECER seismic research program has placed increased emphasis on existing construction.

Recent Research Efforts

Seismic instrumentation plans and preliminary seismic vulnerability assessments for Fort Lewis

This research was supported by CEMP-ET. CECER had earlier performed research projects to develop the procedures outlined in TM 5-809-10-2 for evaluating existing facility vulnerabilities. This project used those procedures to perform a preliminary assessment at Fort Lewis. In the process, CECER surfaced the potential vulnerabilities of unreinforced masonry construction found in many historically significant structures.

Seismic evaluation for Fort Ord

This research was supported by CEMP-FT. CECER had previously performed preliminary vulnerability analyses. In this project, CECER performed detailed seismic analyses on several facilities and developed structural upgrading concepts.

Preliminary seismic analysis of Presidio of San Francisco

This project was also funded by CEMP-ET. CEMP-ET asked that CECER employ a rapid seismic analysis procedure developed

by the Naval Civil Engineering Laboratory to perform vulnerability analyses, both to verify the model and to determine Presidio seismic vulnerabilities.

Seismic instrumentation systems

Funded by CEMP-ET, CECER installed accelerometers in key locations in the hospital facilities at Fort Ord, the Presidio of San Francisco, and Fort Campbell. The 1989 Loma Prieta earthquake triggered the systems at both California installations, providing valuable structural response data. CECER is analyzing the data for its significance in refining structural response models.

Seismic vulnerability analyses of strategic air command bases in California

This was a cooperative project with CEWES and was described earlier in this paper. CECER is producing summary reports of the work in the analyses.

Seismic retrofit techniques for existing concrete buildings

This is an ongoing CERD-funded RDT&E project. The multiyear research program focuses on two separate, but related, areas. The first is base isolation technology in, primarily, retrofit applications; the CECER goal is to develop a standard triaxial testing device and associated test procedures for off-the-shelf base isolation hardware. There is no current standard procedure for designers to use in employing base isolation, a situation that has discouraged its use. Parallel with this is a major research effort to develop appropriate retrofit techniques for nonductile concrete frames, which are inherently vulnerable to seismic motions. The Army and Air Force have hundreds of buildings of this type that were constructed before the mid-1970's; particularly in seismic zones 3 and 4, they are very vulnerable. Current research centers on testing beam-column subassemblages on the BSTM to understand their actual behavior characteristics; follow-on research will examine retrofit techniques that minimize intrusion on building functions, such as viscoelastic damping mechanisms. CECER is performing the research jointly with the University of Illinois, which has received funding from the National Center for Earthquake Engineering Research for its efforts on the project and to construct some of the test specimens.

Biaxial response relationships of concrete frame systems

This is an FY91 new start project that has two primary thrusts. Both the Army and Air Force have many concrete frame buildings, many of which have structural masonry infills that significantly modify frame responses to earthquake motions. The first thrust is to analyze via scaled testing on the BSTM both the in-plane and out-of-plane behavior of masonry infills. CECER is performing this work jointly with the University of Illinois. While CECER will perform the dynamic tests, the University is performing full-scale static tests of the same systems; that work is funded by the National Science Foundation. The other major thrust in this project is the definition of torsional behavior in nonductile concrete frame facilities typical of older military construction and subsequent development of optimization techniques for placing retrofit systems that minimize torsional responses.

Shear strength of multiwythe masonry walls

This project examines the inherent in-plane shear strengths of multiwythe load-bearing brick wall systems through test specimen testing on the CECER structural load floor. Such walls are typical in historical facilities found at Fort Lewis and other posts, and they form the primary lateral force resisting systems in the facilities. Preliminary indications are that the inplane shear strength of these systems does not vary linearly with the number of wythes present. This project will better enable strength assessments of existing facilities to be made. As a follow-on to the initial project, CECER will repair the test samples and reload the walls to analyze the effectiveness of existing repair techniques.

Improved rapid seismic analysis procedure (RSAP)

This is a recent new start project in the centrally funded Small Business Innovative Research (SBIR) program. The existing RSAP, which is recommended by TM 5-809-10-2 for evaluating existing structures, was developed by the Naval Civil Engineering Laboratory in the early 1980's for use on microcomputers. Both microcomputer technology and structural engineering have made significant advances since then. CECER experience with seismic vulnerability assessments shows the existing RSAP model provides inadequate definition of structural response. This project focuses on providing current technology analytical capabilities for simplified analyses that indicate whether potential structural damage justifies more rigorous dynamic nonlinear analyses for thoroughly assessing existing facilities.

Use of shape memory alloys in active seismic control of building structures. This project will be a late FY91 new start project in the SBIR program. In this project, CECER will explore the use of rare-earth alloys that exhibit magnetostrictive characteristics (i.e., they change their lengths by large amounts when subjected to electric currents) in building structures. Such materials may be fabricated into secondary frame member inserts to alter building story lateral stiffnesses on demand, thus altering dynamic responses to seismic ground motions. Basic research has shown these materials to exhibit large current-induced strains, rapid mechanical response, low hysterisis, variable elastic moduli, and high electrical efficiency. Initial studies will center on the materials themselves; later research will include appropriate control mechanisms.

CECER has set a number of ambitious goals for itself for the upcoming years

First, in its multiyear CERD-supported RDT&E plans, seismic upgrading of existing facilities receives increased emphasis; with the dominant military construction materials being reinforced concrete (frames) and masonry, the research program will emphasis them. Second, CECER is seeking multiagency federal support for constructing a base isolator test facility that could be used to develop national standards for base isolator implementation in construction. Third, CECER is working with the UIUC, the University of Michigan, the University of Minnesota, and the University of Texas at Austin, to develop a central US consortium for building seismic research; the consortium will emphasize cooperative research initiatives and key on experimental research involving the BSTM in an upgraded triaxial configuration. This will provide the opportunity to leverage limited USACE research funding with that of the National Science Foundation and others. Last, and perhaps most significant, CECER-EM-E is working to develop closer ties to the practicing engineering communities, both within the Army and without. CEMP-ET has recently requested a proposal from CECER-EM-E to develop an Army-wide plan for compliance with the new public law; this work should expand CECER-EM-E contacts from those already in place (CEMP-ET, CEHSC, CESPD, CENPD, CEMRD and CESPK) to include other key players. Since the building construction found on military installations is not peculiar to the military, CECER-EM-E is broadening its federal agency contact base through closer involvement with members of the Interagency Committee on Seismic Safety in construction; prominent partners include the National Institute for Standards and Technology, the National Science Foundation, the General Services Administration, the Department of Veterans Affairs, the State Department, and our sister services, the Air Force and Navy. Key proposed research projects include:

Seismic retrofit techniques for existing concrete buildings

The technical aspects of this project were described previously. The ultimate goal of the project is to develop comprehensive structural upgrading approaches for existing nonductile concrete frame facilities that provide adequate, economic seismic safety but are not disruptive to facility functions. Ongoing project, with anticipated completion in FY94.

Biaxial response relationships of concrete frame systems

The technical aspects of this project were described earlier in the paper. This project is a 6.1 (basic) research initiative. It will transition into more applied research that will develop sufficient characterization of masonry infill responses to in-plane and out-of-plane loads and of reinforced concrete frame behavior under combined in-plane/out-of-plane motions to develop upgrade design criteria. Ongoing project, with anticipated completion in FY93.

Repair/strengthening of unreinforced and underreinforced masonry

With the predominance of older masonry construction on military installations, suitable means of strengthening masonry wall systems, both as preearthquake strengthening and as post-earthquake repair, are needed. This project will assess experimentally the adequacy of proposed strengthening techniques and develop guidance for field applications. Proposed for FY92-FY94.

Seismic vulnerability assessment of nonstructural components

Most vulnerability research efforts center on structural systems. This project will examine critical nonstructural items in typical facilities that could cause injury or loss of life if they fail in an earthquake. The study will examine life-support systems in hospitals for their vulnerabilities and the dynamic response characteristics of key facility hardware (e.g. suspended ceilings, ductwork, architectural details, etc.) through analysis and experiment to develop procedures for assessing system seismic vulnerabilities. Proposed for FY93-FY95.

Decision matrix for seismic upgrading

With the anticipated emphasis on upgrading existing facilities in the future, more adequate means of performing cost-benefit analyses for different upgrade options are needed. This project will develop more accurate means of assessing all costs associated with proposed upgrade schemes, both actual monetary costs and intangible costs (e.g. mission disruption). Proposed for FY95-FY97.

Seismic protection for nonstructural components

This project will be a successor to the assessment project already described. Once vulnerabilities are identified in an existing facility or during the design of a new facility, appropriate mounting and internal structure details for nonstructural components are required. This project will employ both analytical and experimental means to develop less seismically vulnerable systems, ultimately resulting in design guidance for field application. Proposed for FY96-FY98.

Retrofit of precast concrete wall system connections

This project will develop, analyze, and test strengthening schemes for existing precast wall panel connections that will permit substantial seismic motion and associated structural ductility. Many existing tilt-up panel structures on military installations are poorly detailed in the regions where the panels connect to primary framing systems, potentially leading to catastrophic failures. Proposed for FY97-FY99.

Conclusion

In the decade ahead, seismic engineering is likely to assume a more important role in USACE design and construction activities, both because of growing life safety concerns and because of legislative initiatives. While most new construction will not pose significant challenges simply because building codes and associated USACE design guidance now include significant seismic design requirements, the large inventory of existing military and civil works structures built before the 1970's code changes will require examination and possible upgrading. CECER and CEWES

have significant research capabilities that can be focused on the challenges, both individually and in concert, and both laboratories have proposed major research programs to address some of the more significant shortcomings in current USACE procedures. The decade poses a unique opportunity for CECER the CEWES, and other USACE partners, to work together in solving and nation's infrastructure problems, both in the seismic field and others.

Overview of CPAR/REMR

by
William E. Roper[1]

Overview of the First Three Years of the Construction Productivity Advancement Research Program (CPAR)

Products are now being developed and transferred into application from the first two years of the program. The selection of the next group of partners for the FY91 program is almost completed. These innovative technologies will have a significant impact for improving productivity in the construction industry.

The 1989 program consisted of 16 projects, utilizing $2.73 million from the Corps and $9.09 million from industry partners. In FY90 the Corps provided $3.828 million as its contribution to the cost-shared, cooperative R&D effort. Construction industry partners contributed $5.155 million, for a total program of $8.983 million. Average duration of the projects is two years.

The 13 projects in the FY90 program involved 22 firms and organizations and were selected from more than 180 R&D proposals made to Corps laboratories. The CPAR Executive Committee, made up of senior Corps headquarters executives, selected the final 13 projects using the CPAR *Criteria for Evaluation,* which emphasizes benefits to construction productivity and Corps missions, and includes other actors such as ease of adoption and technology transfer, chance of success, project duration, and cost.

The FY90 projects included an innovative asphalt repaving process, development of light-weight concrete masonry units, improved design procedures for masonry construction, evaluation of improved concretes, including the use of recycled thermoplastics, and processes for bioremediation of hydrocarbon-contaminated soils and groundwater. Substantial benefits are expected from the project when put into use in the construction industry, ranging from 20 percent or greater reductions in cost to reduction in job-site injuries.

The organizations participating in the FY89 and 90 programs are a cross-section of the construction industry, and include engineering firms, equipment and material manufacturers/ suppliers, trade and professional groups, academic institutions, state and local agencies, and utility companies.

REMR: Summary of Accomplishments and a Look to the Future

The primary objective of the Repair, Evaluation Maintenance, and Rehabilitation (REMR) Research Program was to identify and develop effective, affordable technology for maintaining and extending the service life of existing Corps civil works structures. Although Corps needs were the driving force behind the research conducted, much of the results have application outside of the Corps. Accordingly, another of the Corps' objective was to make REMR research results available to other Federal agencies, state and local governments, and the private sector. A comprehensive technology transfer program was developed to publicize research results to internal and external audiences.

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The REMR Research Program as originally planned was successfully completed in FY89 on schedule and within budget. It has clearly demonstrated the benefits of research in getting more value for the dollars spent on REMR activities. During the final year of the research program, savings in excess of $40 million were attributed to the use of REMR-developed technology. These saving were from first time uses of only a portion of the technology produced and did not include other benefits such as improved safety and reliability, reduced operational manpower requirements, and improved operational capabilities. Additional savings and benefits accrue each time the technology is used and with estimated savings over the next five years exceeding $200 million.

Because of the successes of REMR, the high demand for its products both within and outside the Corps, and the opportunity for similar successes on other REMR-type problems, we initiated a REMR-II Research Program. We expect the program to continue for seven years with an added cost of $35 million. During this time frame, REMR-II will address new and different needs which have been identified by Corps field offices. It will concentrate on problems and areas which have the potential for large payoffs and widespread application.

A few examples include studies of new concepts in maintenance and repair of concrete structures, precast concrete for repair and rehabilitation of structures other than lock walls, evaluation of new repair materials, and nondestructive evaluation systems for diverse applications. Studies will also be initiated on levee rehabilitation and the effects of vegetation on levee reliability, new cost-effective reservoir shoreline erosion control procedures, new developments in seismic stability analysis, acoustic emissions monitoring for geotechnical applications, and methods to reduce rock erosion in spillway channels.

Methodology for a Reliability-Based Condition and Evaluation of Navigation Structures

by

Dr. Mary Ann Leggett[1]

Abstract

A regional assessment of future modernization needs by Corps of Engineers Division offices with significant inland navigation missions was requested in 1989 by the Director of Civil Works, US Army Corps of Engineers (HQUSACE). Due to the limited amount of funding available for maintenance, rehabilitation, or replacement of aging navigation structures, these assessments would aid in forming a nationwide planning and priorization system. In 1990, the Inland Waterways User's Group began formulation of this information into an economic traffic-based national assessment model, General Equilibrium Model (GEM). As work on GEM progressed, the user's group realized that a navigation structure's condition should also be considered. In the final months of fiscal year 1990, the Structures Branch of Engineering Division at HQUSACE brought together a multi-discipline team to determine the feasibility of modeling a navigation structure's condition. This team established the basis of a methodology for a reliability-based engineering assessment model for navigation systems. The purpose of this paper is to present the findings of the multi-discipline team and to review progress to date on this reliability-based method.

Introduction

The condition of the Nation's navigation system has a widespread effect on the country's economy, as its cargo includes agricultural, petroleum, and forest products, coal, crude oil, and industrial chemicals. In addition to carrying over 15 percent of the Nation's intercity freight, transportation by water impacts international commerce by carrying exports to coastal harbors. Like all other transportation routes, there is a tremendous expense associated with delays or suspension of operation on the waterways.

The infrastructure of the inland navigation system is currently deteriorating due to aging of the system's structures. Many of these structures have exceeded their design life and/or capacity, and major maintenance work is necessary to keep them operational. As each structure ages, its maintenance cost increases. Locks 1 and 2 on the Kentucky River are approximately 150 years old, while Lock and Dam 26 (Melvin Price) is less than 3. Although the median age is approximately 37, more than 40 percent of the structures are over 50 years old. Fifty years was the design life for most structures. Performance below satisfactory levels could occur at any time, further straining an overloaded system. An additional consideration is that the structures in a particular waterway were generally built

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at the same time, compounding the situation for Corps Divisions such as the Ohio River and Lower Mississippi Valley Divisions. Also, generally the size of the currently used tows is greater than that for most of the older locks.

Major capital requirements will be needed for rehabilitation and modernization of the inland navigation system. Presently, revenue for major projects is provided by allocation in the Federal budget and by usage of the fuel taxes placed in the Inland Waterways Trust Fund. Available funding will not be sufficient to meet all of the system's requirements.

Solution Objective

A long-term investment strategy is needed for identifying and prioritizing the critical needs within the inland navigation system. When considering investment decisions for navigation structures, three main areas need to be addressed:

- Planning for future rehabilitation work.
- Current operation and maintenance work.
- Design for future modernization needs.

Therefore, the purpose of this initial study was to establish a methodology for determining an engineering systems performance or the likelihood (probability) of a malfunction occurring. Then, this engineering condition-based evaluation can be used in the determination of the optimum course of action for the decision problem under the restriction of limited resources. Additionally, it would provide a consistent means of comparing the relative condition of different components of a structure, the relative performance of alternative designs, and the overall condition of different structures.

Reliability Method

Reliability is defined as the mathematical probability that a system will operate as required. Methods of reliability have been used to study the lifetime of systems and to relate this lifetime to factors that determine performance. Civil engineers have begun to use reliability analysis to permit the reasonable quantification of the expected condition of a system. The recent Load and Resistance Factor Design (LRFD) method for steel structures utilizes reliability methods. For probabilistic analysis, parameters used in the calculations are treated as random variables, where they are represented by probability distributions rather than explicit values. Results from this probabilistic analysis may be expressed as a reliability index.

Traditionally, the safety of structures and their components is measured in terms of a factor of safety (FS). For any performance mode, the factor of safety is expressed in the form

$$FS = C/D \qquad (1)$$

where C = capacity function and D = demand function . This capacity function could be strength or ultimate stress, while applied load or applied stress is represented by the demand function. When *capacity is less than demand*, a structure will perform unsatisfactorily. The limit state for FS is achieved when capacity equals demand or

$$C/D = 1 \qquad (2)$$

In deterministic analysis, the components are designed such that the ratio in Equation 2 exceeds unity by some acceptable minimum value. Therefore, this acceptable value is dependent upon the problem and performance mode being investigated.

In probability analysis, the uncertainty in capacity and demand can be expressed by a probability distribution for each variable. Figure 1 depicts general capacity and demand distributions and their limit state. These probability distributions are constructed by allowing one or more of the independent variables from a deterministic analysis to be random variables and performing the calculations using the random variables rather than a single value. Then, the probability of unsatisfactory performance $P(u)$ is

$$P(u) = P(C < D) = P(C/D < 1) \tag{3}$$

and the reliability R is

$$R = 1 - P(u) \tag{4}$$

Usually the distributions of capacity and demand are unknown, since the distributions of the input parameters on which they are based is unknown. But if the mean μ and standard deviation σ of the input parameters are known, then the mean and standard deviation of capacity and demand may be determined. Reliability may then be expressed as a function of the means and standard deviation of capacity and demand by a reliability index β.

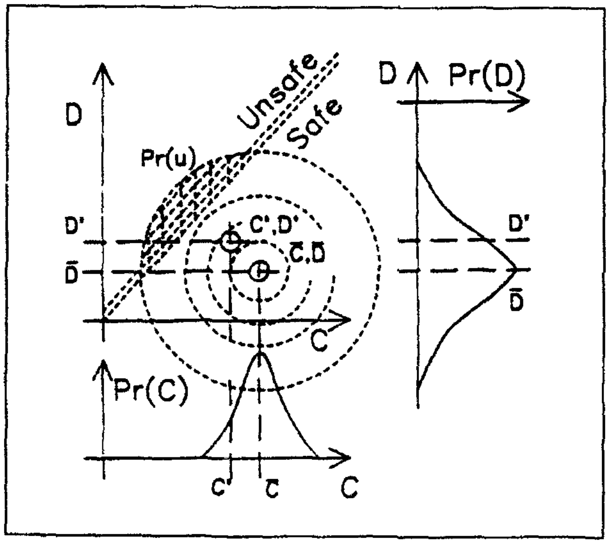

Figure 1. Joint distribution of C and D

The safety margin SM is defined as

$$SM = C - D \tag{5}$$

When its value is less than zero, unsatisfactory performance occurs. Expected value or mean $E[SM]$ and standard deviation, σ_{SM}, of the safety margin may be expressed as

$$E[SM] = E[C] - E[D] \tag{6}$$

$$\sigma_{SM} = \sqrt{\sigma_C^2 = \sigma_D^2} \tag{7}$$

Estimates of the mean and standard deviation of capacity and demand can be derived using simulation methods, Taylor series approximations, or point estimate methods. It can be assumed that capacity and demand are normally distributed, since most distributions can be approximated using a normal distribution. Then, the probability of unsatisfactory performance may be calculated as

$$P(u) = P(SM < 0) = \Phi \frac{-E[SM]}{\sigma_{SM}} \tag{8}$$

where Φ = cumulative standard normal distribution. Equation 8 may be rewritten as

$$P(u) = \Phi(-\beta) \tag{9}$$

where

$$\beta = \frac{E[SM]}{\sigma_{SM}} = \frac{E[C] - E[D]}{\sqrt{\sigma_C^2 + \sigma_D^2}} \tag{10}$$

This reliability index includes information concerning the magnitude and uncertainty in the variables and can provide a consistent means of comparing the performance among different navigation structures.

Lock Wall Example

An example was prepared to illustrate the application of reliability analysis to the problem of sliding of an anchored lock wall. It represented the analysis of multiple random variables, their possible correlation, and the different classes of probability distributions that the variables might assume. The problem is illustrated in Figure 2 with a free-body diagram shown in Figure 3. It consists of a concrete lock wall 20 ft high and 10 ft wide, founded on a competent, jointed limestone and anchored by two rows of anchors. Water surface is at pool level on one side of the wall and at tailwater level on the other side of the wall. Probabilistic methods, using the point estimate procedure, were used to determine

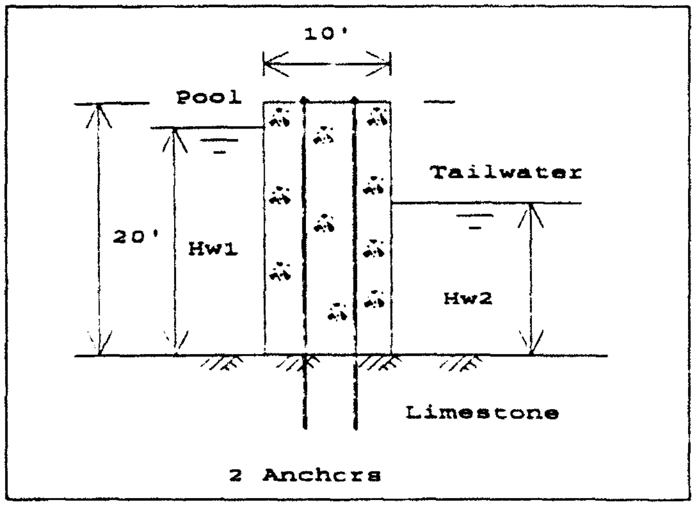

Figure 2. Illustration of lock wall sample problem

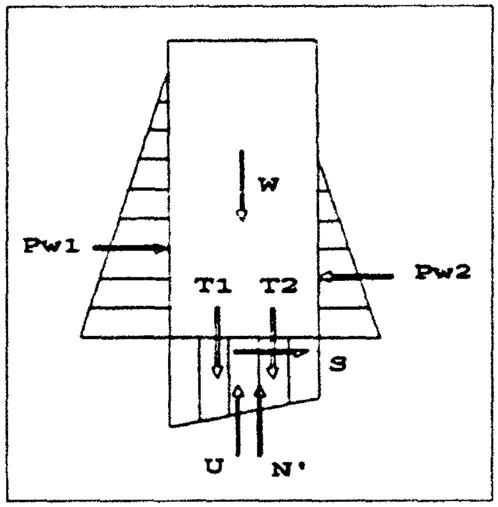

Figure 3. Free-body diagram

the reliability against sliding due to the differential water force.

The weight W is given by

$$W = (20)(10)\gamma_{conc} \tag{11}$$

Headwater P_{w1} and tailwater P_{w2} forces are

$$P_{w1} = (0.5)\gamma_w H_{w1}^2 \tag{12}$$

$$P_{w2} = (0.5)\gamma_w H_{w2}^2 \tag{13}$$

Total uplift force U_E is

$$U_E = 10\gamma_w \frac{2H_{w2} + (1-E) * (H_{w1} - H_{w2})}{2} \tag{14}$$

where E = drain efficiency factor and 10 = width of monolith. T_1 and T_2 are the anchor forces per lineal foot of lock wall. Effective base resultant N' is

$$N' = W + T_1 + T_2 - U_E \tag{15}$$

and the maximum available base shear resistance S_{max} is

$$S_{max} = N'(\tan\phi') \tag{16}$$

where ϕ' = drained friction angle of the limestone-concrete interface. The demand D, causing the monolith to slide, is the difference between P_{w1} and P_{w2}, or

$$D = P_{w1} - P_{w2} = (0.5)\gamma_w (H_{w1}^2 - H_{w2}^2) \tag{17}$$

Capacity C , resisting sliding, is the maximum base shear force S_{max} or

$$S_{max} = (W + T_1 + T_2 - U_E)(\tan \phi') \tag{18}$$

Exact solution for the expected values of capacity and demand are obtained by integrating the function over the probability distributions of the random variables.

$$E[C] = \int\int\int\int\int\int\int (W + T_1 + T_2 - U_E) \tan \phi \, f(W,T_1,T_2,U_E,H_{w1},H_{w2},\phi) \, dw \, dT_1 \, dT_2 \, dU_E \, dH_{w1} \, dH_{w2} \, d\phi \tag{19}$$

$$E[D] = \int\int (0.5) \gamma_w (H_{w1}^2 - H_{w2}^2) f(H_{w1},H_{w2}) \, dH_{w1} \, dH_{w2} \tag{20}$$

Standard deviations are calculated as

$$\sigma_x = \sqrt{Var[X]} = \sqrt{E[X^2] - (E[X])^2} \tag{21}$$

where $X = C$ or D and $E[X^2]$ is evaluated by replacing the functions in Equations 19 and 20 with their squares.

In the point estimate method, the probability distribution of a random variable X is replaced by a discrete probability distribution having only two values X_+ and X_- with probability weights of P_+ and P_-. Values and probability weights are chosen to match the mean and standard deviation for the distribution being represented, and Table 1 summarizes these values for the lock wall.

Table 1
Summarization of Values from Point Estimante Method

Variable	Symbol	Mean	Standard Deviation	X_+	P_+	X_-	P_-
Concrete density (lb/ft^3)	γ_{conc}	145	5	140	0.5	150	0.5
Friction angle (deg)	ϕ	30	5	34.3	0.425	24.18	0.5
Anchor forces (lb/ft)	T			15000	0.9	0	0.1
Headwater (ft)	H_{w1}	14.69	1.71	17.17	0.677	13.15	0.323
Tailwater (ft)	H_{w2}	9.14	1.96	11.25	0.536	7.32	0.464
Drain efficiency (%)	E	0.677	0.234	0.982	0.641	0.535	0.359

A major distinction between navigation and other structures is accounting for the variation of the water levels. In this sample problem, the headwater, or pool, and tailwater levels are not independent. Tailwater cannot exceed the headwater. For any headwater level, the tailwater level is bounded by zero and the headwater level. Therefore, the values for a joint distribution as shown in Table 2 must be obtained by combining the marginals and adjusting for the correlation coefficient.

Table 2
Joint Probability Values for Water Levels

Condition	Headwater	Tailwater	P
+	17.17	11.25	0.5156
+-	17.17	7.32	0.1614
-+	13.51	11.25	0.0204
—	13.51	7.32	0.3026

Having expressed the random variables as pairs of point estimates, numerical integration is used to calculate the expected value. For capacity,

$$E[C] = \sum \left\{[C_{w,H_{w1},H_{w2},T_1,T_2,U_E}] \Pi [P_{w,H_{w1},H_{w2},T_1,T_2,U_E}]\right\} \tag{22}$$

where the capacity function is summed over all combination of the + and - point estimates weighted by the product of their associated p values. The other variables can be evaluated by the same process. Equation 10 is then used to determine β, whose values are given as a function of anchor reliability in Table 3.

Table 3
Reliability Indices

Anchor Reliability	E[C]	E[D]	σ_C	σ_D	Reliability Index
0.90	27269	5174	6812	1240	3.19
0.95	28091	5174	6513	1240	3.46
0.99	28749	5174	6226	1240	3.71

Actual Application

This reliability-based procedure is currently being applied to establish an engineering assessment of Lock and Dams 2, 3, and 4 on the Lower Monongahela River. Four main areas which are being evaluated include:

- Structural stability of gravity dams.
- Steel structures as related to lock gates.
- Concrete deterioration.
- Structural stability of pile foundations.

For each of these four areas, the reliability model is being calibrated using a structure which meets:

- Current design criteria.
- One that has suffered extreme distress.

Then the model will be used to predict the reliability for structures on the Lower Monongahela.

To examine the structural stability of gravity dams, a number of structures which reflect a variety of conditions were selected: monoliths with and without anchors, monoliths with and without earth backfill, dam piers, and structures with limited data versus those with voluminous data. Structures will be analyzed for sliding stability, overturning and bearing capacity, and each mode of performance will be computed under three pool conditions. A comparison of reliability among the selected structures will then be assessed.

Reliability index for a vertically framed miter gate will be defined to be the minimum of the indices for the component members of the gate. Both the old and new miter gates at Emsworth Lock and Dam are being evaluated to provide extreme end points for possible conditions. The evaluations are being based upon statically determinate deterministic models of the individual components. A variety of structural elements under various combinations of loading conditions are being considered. These loadings include those due to hydraulics, boat impact, and water resistance.

The work of determining a reliability index for concrete in navigation structures will address the rate of concrete deterioration, loss of chamber geometry, and overstressing and cracking of relatively thin deteriorated sections. Previous work under the Corps of Engineers' Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Program will be used as a starting point. The first step will be to develop a model for the rate of deterioration of

concrete, resulting in a probabilistic estimate of the annual rate of deterioration as a function of environmental temperature, saturation, operation conditions, and state-of-stress within the concrete. A reliability index representing operational impairment of lock chambers due to loss of geometry from concrete will then be developed. Also, a reliability index for concrete subject to overstressing and cracking in thin sections subject to severe deterioration will be formulated.

The reliability index for evaluating pile foundations will be defined as the minimum of the reliability indices for the overstressing of the individual piles within a pile group. The Corps computer program for pile group analysis (CGPA) will provide the deterministic basis for the evaluations. This code efficiently considers the three-dimensional, statically indeterminate response of a pile group with a rigid cap to static loading. The stresses in the individual piles can then be examined in the determination of the reliability index. Loading will principally involve combinations of dead weight, live loading, hydraulic pressures, and impact loadings.

Conclusion

A reliability-based engineering analysis is preferable to factors of safety as a means of assessing the performance level of navigation structures. While factors of safety relate to the reserve capacity between expected loads and structural failure, they do not directly relate to serviceability or performance. Design values used in traditional analysis of locks and dams are conservatively chosen based on experience and judgment. Probabilistic analysis provides reliability indices based on the average value and variability of the input variables.

The major emphasis of this initial study was to develop a methodology for an engineering assessment of the condition of navigation systems by means of a reliability-based model that is mathematically sound. This work will provide the basis for general criteria-development and supporting PC-based models to assess aging structures. In order for future development of a generalized end-product, the current model will need to be expanded to incorporate a wider range of components and performance modes. Additional work needs to include evaluating more structures, thereby allowing refinement of the curve used to predict reliability and obtainment of a greater confidence level.

Acknowledgments

The author acknowledges the Planning and Engineering Divisions, HQUSACE, Institute for Water Resources, and Civil Works Guidance Update Center, WES, for sponsoring this study. Initial efforts in applying reliability analysis to the navigation structures on the Lower Monongahela were begun by Messrs. Jeffrey Benedict and Bruce C. Riley of the Planning and Engineering Divisions, Pittsburgh District. Dr. Thomas F. Wolff, Michigan State University, helped prepare the sample problem illustrated in this paper.

References

Benjamin, J., and Cornell, C. A. 1970. *Probability, Statistics, and Decision for Civil Engineers*, McGraw-Hill, New York.

Frankel, E. G. 1988. *Systems Reliability and Risk Analysis*, Kluwer Academic Publishers, Boston.

Haviland, R. P. 1964. *Engineering Reliability and Long Life Design*, Van Nostrand, New York.

Rosenblueth, E. 1981. "Point Estimates for Probability," *Applied Mathematical Modeling*, Vol 5, No. 5, pp 329-335.

Investigation of Lift Gate Failure Locks 27, Mississippi River

by
Robert D. Kelsey[1] *and Thomas R. Ruf, PE*[1]

Abstract

In March 1989, severe damage to the upstream leaf of the main lock lift gate at Locks 27 was discovered. A study was undertaken to determine the cause of the damage and to determine what remedial measures should be taken. The study consisted of testing samples of material removed from the lift gate, performing an in-depth structural analysis of the lift gate, and instrumenting the lift gate. The material tests were performed to determine strength, toughness, hardness, and chemical composition of the lift gate material. The structural analysis consisted of an examination of the original computations and a computer analysis utilizing a three-dimensional finite element model of the lift gate. Strain gages were installed to determine actual member stresses. Conclusions from the study concerning causes of failure and a summary of remedial measures taken are presented.

Introduction

Locks No. 27 are located on the Chain of Rocks Canal (which bypasses the Chain of Rocks stretch of the Mississippi River) at Granite City, IL, 185.1 miles above the mouth of the Ohio River. Construction of the locks was completed in 1953. The locks consist of a main lock, 1,200 × 110 ft, and an auxiliary lock, 600 × 110 ft. The lock gates consist of vertical lift gates at the upstream end of the locks and miter gates at the downstream end of the locks. Prior to the addition of a low-water dam at the Chain of Rocks on the Mississippi River in the early 1960's, and a subsequent raise in pool elevation at Locks 27, alterations were made to the lift gates and upper sills to accommodate the higher head caused by the addition of the low-water dam. The low-water dam was constructed to increase the depth of water over the downstream miter gate sills at Locks and Dam 26, which did not provide adequate draft for loaded tows at low river stages.

Each lift gate consists of two welded structural steel leafs which span the width of the lock chamber (110 ft). Each leaf is 30 ft high. A skin plate on the upstream side of each leaf forms the vertical damming surface. Plate girders transfer horizontal loads acting on the skin plate to the reactions at the lock walls. The top girder of the upstream leaf forms a horizontal damming surface, with pressure from upper pool acting on the top surface and lower pool pressure acting on the bottom of the girder. The bracing on the downstream side of the leaf between the girders forms a truss to support vertical loads. Sealed buoyancy chambers are intended to be watertight and provide a reduction in vertical load. Chains and associated machinery, located in recesses in the lock walls at each end of the gate, provide means for

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adjusting the elevation of the leafs. The upper leaf (downstream leaf) is operated (lowered and raised) for each lockage. The lower leaf (upstream leaf) is infrequently operated, to adjust for varying pool elevations (see Figure 1 for gate geometry).

Description of Damage

In March 1989, a construction contract was underway at the project to make repairs to the lock wall embedded metals. The lock was partially dewatered, and both lift gate leaves were out of the water to provide clearance for the repair work. Under normal conditions, the upstream leaf is underwater and is not visible. An inspection of the gate revealed cracks in the bracing and girders on the downstream side of the upstream leaf of the main lock lift gate. This damage was serious enough to warrant immediate repairs to prevent a possible catastrophic failure of the gate. Twenty-four braces between the downstream flanges of the girders were completely fractured. Twenty-two braces were partially fractured. Six girder flanges had cracks that extended through the flange and into the girder webs 3 to 42 in. At two of these locations, the cracks also went through the girder flange cover plate. Seven girder flanges were partially cracked. Eleven girder/end plate connections were cracked with cracks varying from 1 to 12 in. Both ends of the upstream flange of the top girder were bent over in the downstream direction. Additionally, all buoyancy chambers were found to be filled with water.

Initial Repairs

It was decided to complete repairs to the lift gate with hired labor as soon as possible so that the lock closure period (in effect at the time for the embedded metals repair contract) would not have to be extended. The repairs were based on the original and alteration designs, with the intention of performing an in-depth structural

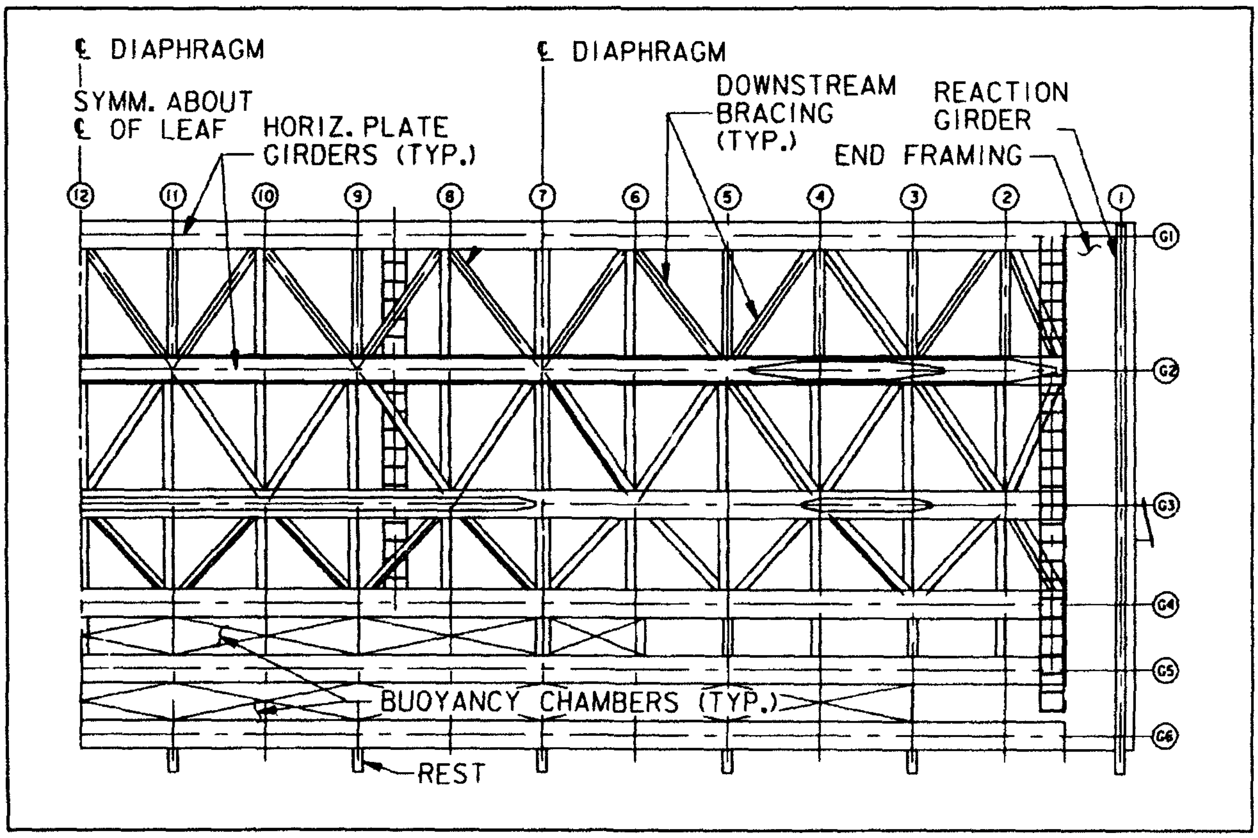

Figure 1. Lift gate upstream leaf

analysis at a later date with possible modification at that time. The repair work involved several operations. The gate was sprayed off with water to remove loose debris, then sandblasted at critical locations to facilitate inspection. The lock was dewatered, and the gate was placed on its supports at the bottom of the lock chamber. The buoyancy chambers were drained and resealed.

Cracks which had not progressed all the way through members were gouged out and rewelded. Braces which were completely fractured were removed and replaced with members of equivalent cross-sectional area. Gussets were used to connect braces to the girder flanges wherever possible to facilitate construction and improve fatigue resistance. Welds were designed assuming the braces were all carrying the maximum allowable load.

Cracks partially through girder flanges were gouged out and rewelded. The crack tips in girder webs were found using dye penetrant. The cracks were arrested by drilling a 1-in.-diam hole at the crack tip to remove the crack tip and some uncracked base metal. The hole at the crack tip was left open. At locations where the crack went through the cover plate and girder flange, the cover plate was removed for a distance of 18 in. on either side of the crack in the girder flange. A new cover plate was fabricated to length and contained edge preparation for splicing into the cover plate. The new cover plate was lapped over the flange so the weld butt joints were staggered to enhance crack resistance.

Cracks through the end plate at the end of the girders were gouged out and rewelded. Cover plates were put over the end of the girder flange/end plate connection.

Follow-Up Inspections and Investigations

Initially, it was thought that the damage to the main lock lift gate was due to the ineffectiveness of the buoyancy chambers and the resulting additional vertical load. Later, it was revealed that the lift gate had been hit by a tow in 1986. An inspection after the accident indicated damage only to the top girder flange. The tow rode up on top of the leaf and had to be backed off under power. It was surmised that the large vertical force from the weight of the tow could have caused the damage to the bracing. If this was the case, the auxiliary lock lift gate should not have any damage as it had not been hit by a barge. An inspection of the upstream leaf of the auxiliary lock lift gate was made after repairs to the main lock lift gate were completed by hired labor. Some damage was discovered, but the condition of the gate was not nearly as serious as the condition of the upstream leaf of the main lock lift gate. The locations of the cracks found on the auxiliary lock lift gate were similar to the main lock; therefore, it was presumed that the source of the damage was the same for both and that the cause of the main lock damage was not barge impact alone. However, this could have been a reason the damage to the main lock lift gate was more severe than the auxiliary lock. Another possible reason was that the main lock had been used much more than the auxiliary lock and therefore experienced more loading cycles on the gates and higher fatigue.

After completion of the repairs, two phases of additional analyses and testing were planned. Phase I consisted of material testing, review of the original and alteration structural computations, and additional inspections. The plan was to periodically inspect the gates until the cause of the damage was identified and remedied. Phase II consisted of an in-depth structural analysis of the gate considering all available information and utilizing a three-dimensional computer model of the upstream leaf.

Several possible sources of the damage were identified for consideration. The following were considered significant enough to have warranted further investigation:

- **Ineffectiveness of buoyancy chambers**. The gates were designed for a reduction in the dead weight due to buoyancy from

the air chambers located near the bottom of the leafs. However, the buoyancy chambers were found to be flooded; therefore, the vertical load on the gate was higher than what was assumed in the original design.

- **Fabrication**. Many of the welds were undercut, which reduced the cross-sectional area of the bracing and caused stress risers and susceptibility to cracking. Also, no evidence of low hydrogen welding practice (such as preheating, low hydrogen electrode selection, and post weld preheat) was found by examining the plans, specifications, and construction photographs from the original construction and alteration contracts.

- **Connection details**. Some of the bracing was eccentric with the assumed panel points, which resulted in secondary stresses due to bending which were not considered in the original design. The alterations made in 1960 also induced an eccentricity into some of the connections. The girder flange/brace connection detail also causes a stress concentration to occur.

- **Design deficiency**. The assumed load paths, method of analysis, and controlling loading conditions were examined to determine if there was a design deficiency in the original design or alteration design.

- **Defective material.** Samples of material from the gates were retained and tested to determine if the gate distress was due to defective material.

- **Operational considerations**. The operating manual was reviewed to determine if the operating procedures stated in the manual matched the design computation assumptions. The actual operating procedures used by the lock personnel were also investigated to determine if the gate was being operated according to the operating manual.

Examination of Computations

The original and alteration design computations were obtained and reviewed to determine what assumptions were made in the design so the assumptions could be compared with the actual operating conditions.

Original design method

The bracing was designed to carry vertical loads. Vertical water loads and dead loads (which included ice and mud load) were assumed to be divided equally between the skin plate on the upstream face and the bracing on the downstream face. Horizontal loading is transferred from the skin plate to the horizontal girders. It was assumed that the three vertical diaphragms prevented differential loading between girders and caused the gate to deflect uniformly in the horizontal direction. It was also assumed the downstream bracing served to prevent local buckling of the downstream girder flanges as well as support the vertical load.

The bracing was assumed to act as five separate trusses, stacked on top of one another. Each truss was assumed to carry a portion of the total vertical load, the portion being the ratio of the panel height to the total leaf height.

Buoyancy chambers located in the bottom two truss panels were designed to be filled or emptied to vary the total buoyant force from 46 to 55 percent of the total gate weight. The loading cases considered the buoyancy to be either 0 (chambers ruptured) or 50 percent of the total gate weight. In the cases where 50-percent buoyancy was considered, the buoyant force was considered to be evenly distributed about the structure. The gate was designed for 8.2-ft head while supported on the chains and 21.2-ft head while supported on the rests.

Design method for alterations

The designer of the alterations in 1960 disagreed with the original designer's assumption on the distribution of the loading on the trusses. It was stated that in order for the distribution to take place, verticals (6 × 6 angle 3/8 thick) must be installed. Once the verticals are in place, the loading distribution would have been as originally assumed. Almost all of the

vertical downstream bracing members were added during the 1960 alterations.

The designer of the alterations used the original loading cases except for the revised water head (due to the addition of the low-water dam) and additional dead weight due to the alterations. The revised heads were 17.3 ft for the gate supported on the chains and 23.0 ft for the gate supported on the rests.

Material Testing

Two sets of material tests were performed on material removed from the upstream leaves of the main and auxiliary lock lift gates.

Description of first set of testing

The first set of tests was performed on material removed from the main lock lift gate. Samples of material from the original construction contract and alteration contract were tested. Plate, angle, bar, and weld material were tested. The following tests were performed:

- **Charpy V-notch**. Charpy V-notch tests provide an indication of a material's ability to absorb energy, which is directly related to toughness (a material's ability to resist crack propagation). Coupons from the plate, bar, and angle were tested at 70, 55, 40, and 32 deg Fahrenheit, in both the longitudinal and transverse direction.

- **Tensile**. Tensile tests to determine yield strength, ultimate strength, and percent elongation were performed on the plate, bar, and angle material.

- **Chemical analysis**. Chemical analyses (measuring the percentage of the following 10 elements: carbon, manganese, nickel, chromium, copper, molybdenum, vanadium, sulfur, phosphorus, and silicon) of the plate, bar, angle, and weld material were performed. These analyses provided information that was used to evaluate two aspects concerning the weldability of the material. The first was the susceptibility to underbead cracking (cracking at the base of a weld). The second was potential for heat-affected zone cracking (cracking in the base metal adjacent to the weld).

- **Micro hardness survey and Brinell hardness.** Two micro hardness surveys were performed on the weld samples. Brinell harness tests on the plate, bar, and angle were performed. Brinell hardness tests, along with micro hardness surveys and chemical composition tests, provided information to evaluate the susceptibility to cracking.

Results from first set of testing

- **Charpy V-Notch**. The data from the testing for the angle indicated that this material had poor toughness compared with historical data for similar material (Barsom and Rolfe 1987). The low values were originally thought to be attributed to the size of test coupons, which were limited to the thickness of the angle material and were smaller than the standard size coupon on which the historical data were based. The data for the bar tended to show lower energy absorption at higher temperatures when compared with historical data. The values for the plate were considerably lower than the historical test data.

- **Tensile**. The testing indicated the yield strengths for the angle and bar exceeded the minimum required by ASTM A36. However, the plate yield strength was 29,733 psi, which was well below the minimum requirement of 36,000 psi.

- **Chemical analysis**. Based on the chemical composition tests, the base materials did not appear to be highly sensitive to either underbead cracking or heat-affected zone cracking. The chemical analysis for the fillet weld indicated a high carbon content (0.17 percent) relative to the chemical composition for typical electrodes (0.06 to 0.08 percent for E60 electrodes, 0.08 to 0.12 percent for E70 electrodes). The weld metal carbon content would have increased slightly by picking up carbon from the

base metal during welding, but even considering this, the carbon content was high.

- **Micro hardness survey and Brinell hardness.** The test data showed that the angle bar and plate all had hardness values which were below the maximum suggested limiting values to assure satisfactory performance against underbead cracking and heat-affected zone cracking, thus indicating the material was satisfactory in this respect.

Conclusions from first set of testing

Based on a limited number of tests, definite conclusions concerning the quality of the materials could not be drawn at the time. However, there were indications that the material may have been deficient in several areas. The Charpy test values were low, indicating poor toughness. The tensile strength of the plate was also somewhat low. The carbon content of the weld metal was slightly high, which would reduce ductility and promote cracking. Because a limited number of tests were performed and there was a wide scatter of data, and the data appeared to indicate the material was deficient in some areas, it was decided to perform additional testing. Some of the existing material that had been removed during the emergency repair contract was retained for this testing.

Description of second set of testing

Additional tests were performed primarily because the previous tests revealed that the toughness of the material appeared to be low. Other concerns were that the carbon content of the weld metal was high and tensile strength of the plate material was low. The second set of tests was performed on material removed from the upstream leafs of the main and auxiliary lock lift gates. Plate material was tested to verify previously measured Charpy values (which were low), carbon content (which was high), and tensile strength (which was low). Charpy V-notch coupons were tested at 0, 32, 40, 55, and 70 deg Fahrenheit. Only transverse Charpy tests were performed. Charpy tests were done on a sample of angle material used for repairs made by hired labor on two different specimen sizes to determine how specimen size affected the test results, so that this information could be used to determine if Charpy tests done on the original angle material (from the original construction of the gate) were low due to specimen size (7.5 × 10 mm sub-size versus 10 × 10 mm full size) or were actually due to low toughness. A fracture analysis of a crack located in the angle was performed to determine additional information concerning how the crack developed. Charpy tests were performed on the material used for replacing some of the braces in the repair contract. The mill test reports furnished by the contractor indicated the material had adequate tensile strength. Charpy and tensile tests were performed on a portion of girder flange plate (from the original construction) removed from the upstream leaf of the auxiliary lock lift gate. Chemical analyses of a weld were performed to verify the previous test which indicated high carbon content.

Results from second set of testing

- **Charpy V-notch tests**. The values for the sub-size test coupons were slightly lower than the values for the full size coupons, but the difference was not significant. The previous tests of the angle material indicated low Charpy values (using sub-size coupons). It was concluded that the low Charpy values from the first set of material tests for the angle material were not due to specimen size as originally thought but rather low toughness. The data for the plate material indicated that this material (from the original construction contract) also had poor toughness relative to historical test data. The data for the plate material agree with the previous testing and indicate that this material (from the alteration contract) has very poor toughness relative to historical test data. The data for the replacement angle material (from the 1989 repairs) and the WT material (from the 1990 repair contract) indicated that these materials had adequate toughness.

- **Mechanical tests**. The yield strengths (which were again somewhat low) for the

plate material verified the previous testing. The average plate yield strength was 34,745 psi and the lowest strength was 29,893 psi, both of which are below the minimum requirement of 36,000 psi.

- **Chemical composition tests**. The carbon content for the plate (0.28 percent) exceeded the allowable ASTM A36 limit of 0.25 percent. The carbon content from the previous tests from November 1989 (0.27 percent) also exceeded this limit. As carbon content increases, a material will tend to behave brittlely. However, even though the carbon content was high, the elongation for 2 in. (28 and 29 percent from the tensile test) still exceeds ASTM A36 requirements (23 percent for 1-1/4-in. plate, the thickness of the sample) indicating that adequate ductility was provided. The manganese content (0.42 percent) was below the ASTM A36 range of 0.80 to 1.20 percent, which helped explain the low yield point. The carbon equivalent for the plate (0.36 percent) was still below the upper limit considered critical for underbead cracking or heat affected zone cracking. The weld metal carbon content (0.15 percent) was slightly lower than the carbon content measured during the previous testing (0.17 percent), but was still rather high. As stated earlier, the high carbon content of the weld metal reduces ductility and promotes cracking.

- **Fracture analysis**. The fracture analysis revealed that the fracture was of a fatigue nature, due to a one-way bending load, low-to-moderate overload, in a high-stressed concentrated area. The 8-1/2 grain size reported from the etched cross sections indicated a fine grain size microstructure. In general, a grain size of 0 to 5 would be considered coarse and be an indication of poor toughness. A 6 to 10 grain size would classify the material as fine grained, which would be preferable in terms of toughness. This agreed with the Charpy test values for this material which showed the material installed during the repairs made by hired labor had good toughness.

Conclusions from second set of testing

The tests showed that the Charpy V-notch test values for both the original and alteration plate and angle material were low. The low toughness values indicate that once a crack initiates, these materials will have little ability to resist crack propagation. The plate yield strengths were also slightly low, but the ductility of this material appeared good, and the ultimate strength was adequate. The new material added during the repairs made by hired labor and during the emergency repair contract appeared to be adequate in terms of toughness and strength.

Computer Analysis

An in-depth structural analysis of the gate considering available information and utilizing a three-dimensional computer model of the upstream leaf was conducted. The purpose of the analysis was to analytically determine member stresses.

A three-dimensional finite element model utilizing the computer program GTSTRUDL (GTICES Systems Laboratory 1985) was used to analyze the leaf. Bending and stretching (6 degrees of freedom) elements were used to represent the skin plate, girder webs, buoyancy chambers, end framing, and reaction girder web. Beam elements were used to represent the girder flanges, skin plate intercostals, downstream bracing, chain girder, reaction girder flanges, and apron braces. The model consisted of approximately 600 nodes and 1,300 elements.

Two support conditions were modeled: on the chains and on the rests. When the gate is supported on the chains, the gate is supported vertically at the midheight of the leaf on the chain girders at the ends of the leaf. The supports in the horizontal direction are continuous along the downstream edge of the end framing on each side. For loading cases with the gate supported on the rests, the same horizontal reactions as those used for loading cases on the chains were used. The vertical supports are located at points beneath the

buoyancy chambers at the extensions of the buoyancy chamber plates.

All internal member connections were input as fixed. Member eccentricities were input where necessary, mostly occurring in the downstream bracing where some joints were not constructed concentric. Joint sizes were also input where appropriate. The downstream bracing frames into 18-in.-deep girder flanges, causing the members to have a significantly shorter length than the actual distance between the modeled nodes. The stiffnesses for the braces were computed taking into account the joint size.

The loading cases originally considered for the computer analysis followed the original design computations for the 1960 gate alterations with some additions. In the 1960's, the buoyancy chambers were deactivated, and the seal at the sill was removed in an attempt to abate vibration of the gate, so load cases for these conditions were added. Originally, it was thought the problems with the gate were due to the 0 percent buoyancy case which was not considered for the loading condition for the gate supported on the chains. The design head for the gate is 17.3 ft for the gate supported on the chains and 23.0 ft when the gate is supported on the rests.

All load cases consisted of a vertical hydrostatic load on the top girder web, horizontal hydrostatic load on the skin plate (the magnitude depending on the effectiveness of the seal at the top of the sill), buoyant forces from the chambers (the magnitude depending on the state of effectiveness), and dead load of the gate.

The analysis was run numerous times as experiments were conducted with respect to modeling techniques. As a test of behavior of the model, a failure sequence of computer analyses was done. The model was initially analyzed, and the highest stressed member was removed and the model was reanalyzed, and the sequence repeated. If the model had been constructed properly, the failures could be traced one by one and would correspond to the observed failures. The model was revised until a failure sequence produced reasonable results. When members were removed near the ends of the gate, the load increased in members in the same vicinity, the type of behavior that is necessary to produce the failures that were observed.

The controlling load condition was found to be a combination of the case added to account for the removal of the seal at the sill and the case added to account for the ineffectiveness of the buoyancy chambers. For the case of no seal at the sill, the net pressure varies from full net horizontal pressure at the top of the sill to zero net pressure at the bottom of the leaf. Additionally, a great deal of water flows through the 4-in.-wide gap between the skin plate and the sill. Traveling at a high velocity, a decrease in pressure against the skin plate occurs (due to a "Bernoulli" effect), thus lowering pressure on the lower portion of the skin plate even further (see Figure 2). This nonuniform horizontal loading causes a moment about an axis perpendicular to flow to act on the gate, causing an increase in stresses in the downstream braces.

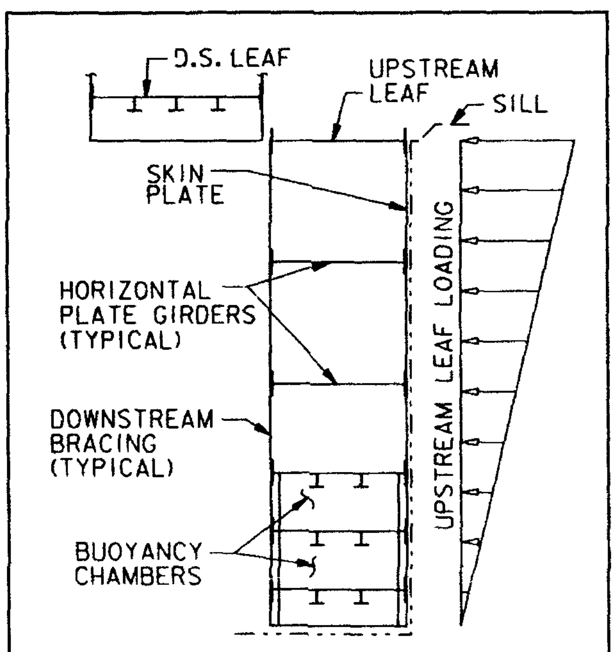

Figure 2. Horizontal water pressure

The results of the analysis using the additional loading cases described above showed improved agreement between overstressed members and members with observed failures, and also improved agreement with strain gage information. The results of the analysis indicate that the leaf undergoes bending in both vertical and horizontal directions, not just horizontal bending as was originally assumed. Bending in the vertical direction causes an increase in compression in the downstream bracing for certain loading conditions (see Figure 3).

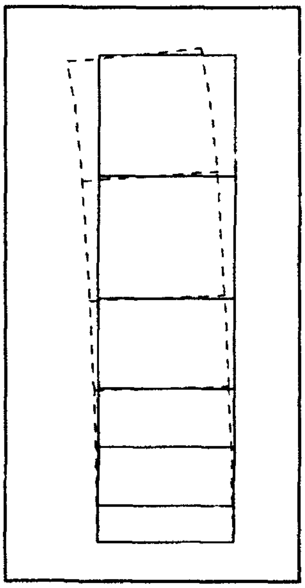

Figure 3. Deflected shape

The following conclusions were drawn from the structural analysis:

- **Improper assumption of load distribution.** It was apparent that horizontal loading of the lift gate has a greater effect on member forces in the downstream bracing than was believed by the original designers. The original designers believed all horizontal loads were distributed to the girders through the vertical diaphragms and the downstream bracing served only to prevent local buckling of the girder flanges under horizontal loading. However, it was clear from the computer analysis, and substantiated by the strain gage testing, that the lift gate acts as a unit under load with the downstream bracing affected by the distribution of the horizontal load. The additional compression in the downstream bracing face due to vertical loads due to water and weight of the gate itself combined with these loads caused an overstress in the downstream bracing members.

- Omission of important load case. The original designers did not consider a load case for the buoyancy chambers 0 percent effective while the gate is supported on the chains. During the gate repairs, many of the chambers were found to be filled with water. The additional vertical load of the water in the buoyancy chambers caused a further overstress in the downstream bracing members.

- **Operating procedures.** It was learned from lock personnel that the lift gate had been routinely supported on the chains for all levels of upper and lower pool due to conflicting information given in the operating manual and tolerances in the gate position indicating equipment. The gate had been supported on the chains for greater hydrostatic head than it was designed for, contributing to high stresses in the downstream bracing as indicated by the computer analysis of this loading condition.

- ***Modeling technique.*** The original designer's assumption of truss behavior of the downstream bracing members is unconservative. Furthermore, the gate was fabricated with many eccentric joints. Both of these items introduce bending moments into the downstream bracing members which further increased the stress. The simplified assumption that the downstream bracing behaved as a truss was made necessary by the crude analysis tools available at that time.

Instrumentation

One set of instrumentation was installed on the lift gate bracing and one set on the lifting chains.

Bracing Instrumentation

Because of the complex nature of the structural analysis of the lift gate, it was felt that some indication of service stress would be helpful in determining the validity of some assumptions concerning the analysis.

The strain gages were installed immediately after the gate repair contract was completed but before the lock was rewatered. Gages were installed on four members of the downstream bracing system. All were WT 9×35.5, newly installed during the gate repair contract.

On each of the members that were selected for receiving gages, three gages were installed, one each near the extreme fibers of each leg of the member. The gage on each leg was placed at the same location along the length of the member, at a reasonable distance from the end of the member to avoid localized effects of connections. In this way, the axial force and bending moment about two axes could be solved for simultaneously and then compared directly to the computer analysis results.

The strain gages used were capable of indicating strain in both the longitudinal and transverse directions. This type of gage was chosen because it could confirm the validity of the strain in one direction using Poisson's ratio and the strain in the other direction. Additionally, if a gage failed in the longitudinal direction, the transverse value could be used to obtain the longitudinal strain. A data acquisition unit was used which was capable of continuous monitoring of the strain gages. Thus, continuous member forces were obtained as the gate was loaded and unloaded.

Several sets of gage readings were taken, first under dead load only, then again as the lock was rewatered and eventually put into operation. There were a total of 12 gages (4 members × 3 gages). Because much data were obtained over time, a short computer program was written to convert strains into the force and two moments in each member.

Conclusions from bracing instrumentation

For the original loading cases, the strain gages indicated member forces much higher than those indicated by the structural analysis. Additional strain gage testing on the chains was later performed to determine if the vertical loading assumptions for the computer analysis were accurate (see the results from the chain instrumentation). The strain gages indicated problems with the loading and/or the structural model. These were later investigated and corrected in the computer analysis, and better agreement was obtained.

Chain instrumentation

The strain gage experiment for the downstream bracing members indicated forces greater than those determined from the preliminary computer analysis. Two possible causes for the discrepancy were identified. Either the loading for the structural model had been underestimated, or the model itself was flawed, giving incorrect distribution of internal forces. By placing strain gages upon the chains, the actual vertical load on the gate could be determined.

The strain gages were placed upon the chains after the gate was repaired by contract and placed back in service. The data acquisition equipment was similar to the data acquisition unit used during the previous strain gage experiments except it could not record continuous readings over time. Strain gages were placed upon the chains on both ends of the gate.

Conclusion from instrumentation

The results of the experiment showed a total measured load about 25 percent lower than that obtained by structural analysis. However, there were many factors which could have caused the gage readings to be inaccurate. Taking all of these possibilities into account, it was concluded that the vertical loading on the gate used for the analysis had not been underestimated. It was therefore determined that the differences between the measured forces in the downstream bracing and that determined from analysis were due to the incorrect estimation of horizontal loads, flaws in the structural model itself, or effects of the horizontal load on the downstream bracing.

It was later discovered from the computer analysis that the horizontal loading of the lift gate had a much greater effect on member forces in the downstream bracing than was

believed by the original designers. It was determined that the lift gate acts as a unit and the downstream bracing is affected by the distribution of the horizontal load. The strain gage experiment was instrumental in pointing the investigation in the proper direction to make this discovery.

Repair Contract

During the Phase I and Phase II investigations, after the repairs were completed by hired labor, inspections of the main lock lift gate were made. During the third inspection, 7 months after repairs were completed by hired labor, significant new damage to the downstream bracing was found. An emergency contract to repair the leaf was then prepared for the purpose of repairing and strengthening the leaf. The repair contract consisted of replacing bracing (20 braces were replaced with larger size members), repairing welds, welding cover plates on girder flanges, and weld inspection. The buoyancy chambers were filled with styrene pellets to ensure their effectiveness in the event of leakage. Preliminary results from the gate structural analysis indicated that some members in the downstream bracing were overstressed under certain loading conditions; therefore, replacement brace sizes were increased. High-strength steel and minimum toughness requirements were specified for the replacement material. Inspections made after the repair contract have indicated only minor distress in the leaf.

Conclusions

Based on the results from the material testing program, structural analysis, and other information obtained, at least five factors contributed to the cracking of the gate:

- **Defective material**. The material used to fabricate the gate and the material used for the alterations made to the gate in 1960 both had very poor toughness relative to similar material being produced presently. These materials do not have the ability to resist crack propagation once a crack initiates from overstress or fatigue.

- **Design assumptions**. Some of the original design assumptions concerning load distribution, load cases, and modeling technique were unconservative. This resulted in actual member stresses (as indicated by the computer analysis and instrumentation) higher than those predicted in the original design. In addition, the bracing connection details are rated in a high fatigue category (American Institute of Steel Construction 1989), and no consideration was given to fatigue in the design.

- **Operating procedures**. The operating procedures were such that under certain conditions the gate was not on the supports for some loading conditions, as assumed in the design. This resulted in additional load in the bracing. The limit switches for the leaf have since been reset to account for tolerances in the gate position indicating equipment to prevent the condition from occurring again.

- **Fabrication procedure**. As stated earlier, there was no evidence of low hydrogen welding practice. This is poor practice considering the alterations to the leafs in 1960 were made during the winter months. These practices make the welds susceptible to cracking. Also, many of the welds were undercut, which reduced the cross-sectional area of the bracing and caused stress risers and susceptibility to cracking. Approximately 90 percent of the welds connecting the downstream bracing to the girder flanges were found to be deficient (did not meet AWS bridge specifications) by an independent testing laboratory which performed an inspection as part of the repair contract. Besides undercutting (which was the most common problem found), the welds did not meet AWS profile and porosity requirements. The seriously deficient welds were repaired during the repair contract.

- **Corrosion**. As cracks initiate and begin to propagate, corrosion occurs at the crack tip and reduces the critical stress intensity factor, thus promoting crack growth. Corrosion also causes reduction in the net area of members resulting in increased stresses.

The final District recommendation was to replace the upstream leaf of the main lock lift gate. There are currently no plans to replace the auxiliary lock lift gate; however, the leaf will be periodically inspected.

References

GTICES Systems Laboratory. 1985. "GTSTRUDL User's Manual," Georgia Institute of Technology, Atlanta, GA.

Barsom, J. M., and Rolfe, S. T. 1987. Fracture and Fatigue Control in Structures, Second Edition, Prentice-Hall, Inc., Englewood Cliffs, NJ.

American Institute of Steel Construction. 1989. "Specification for Structural Steel Buildings," AISC, Chicago, IL.

Seismic Structural Analysis of Olmsted Lock

by

Dr. Robert L. Hall[1] *and Tommy L. Bevins*[1]

Abstract

The seismic structural analysis of the Olmsted lock has presented many challenging problems. The structure is a unique W-frame lock, 1,600 ft long and 326 ft wide. The rock outcrop peak ground accelerations are 0.44 g's for the Operating Basis Earthquake and 1.12 g's for the Maximum Credible Earthquake. This structure is supported by a pile foundation. This is the first lock ever to be designed/constructed on a geological site with such a severe earthquake ground motion possible for the design life of the structure. The combination of geometry, foundation, and extensive ground motion has created problems with predicting hydrodynamic loads, structure accelerations, and determination of seismic design forces.

This paper will present procedures used for hydrodynamic loads and describe how these and other loads were used to conduct a response spectrum calculation for the W-frame lock. A procedure for extracting finite-element (FE) response spectrum stresses for developing shears and moments will be discussed. A comparison between an FE analysis and a CW-frame program using beam elements will also be presented.

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Model for Seismic Analysis of Pile Groups

by

Reed L. Mosher,[1] *Robert Ebeling,*[1] *and Paul Mlakar*[2]

Abstract

The determination of displacement of the pile cap and displacements and forces of individual piles is essential to the design of pile foundation subject to both static and/or dynamic loading. Saul's (1968) procedure for the static analysis of pile group has been widely used for determining the pile cap displacements and individual pile forces and displacements resulting from a global three-dimensional loading. The three-dimensional procedure employs the direct stiffness method of analysis and assumes the pile cap to be rigid and the pile-soil system to be represented by a set of linear springs. These assumptions allow the pile foundation to be reduced to a six-degree-of-freedom system. Notwithstanding these simplifications, the procedure yields reasonable results (O'Neill and Tsai 1984) for rigid pile caps.

Saul's pile group analysis procedure was extended to provide a simplified, yet realistic approach for determining the response of pile foundations subjected to seismic loading (Jones, Mlakar, and Mosher 1989). Viscous damping of pile-soil system and response spectrum loading were added to Saul's original derivation of the frequency equation for pile groups subjected to dynamic loading. The formulation of the mass and damping matrices was developed and a modal analysis with response to spectra loading wax implemented.

Since this research work was first published (Jones, Mlakar, and Mosher 1989), a significant amount of additional development and refinement work has been accomplished. The paper will present the new results from this latest research effort which include the addition of the stiffness contribution due to the soil in contact with pile cap, new procedures to determine the contribution to mass matrix provided by the soil under the pile cap based on the displacements of the piles, refinement of determination of viscous damping of the pile-soil system based on the geometry and motion of the system, addition of hysteretic damping, the inclusion of additional degrees-of-freedom to account for the flexibility of the superstructure.

The approach is applied to ten different published studies to demonstrate the validity of the approach and its potential as a design tool. The results of these comparisons will be reported in the paper. Comparisons have been of results between this simplified method and the more comprehensive finite element analysis using computer code FLUSH.

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Design, Construction, and Rehabilitation of Eisenhower and Snell Locks, St. Lawrence Seaway, Massena, New York

by
Reed L. Mosher[1]

Abstract

The St. Lawrence Seaway Development Corporation (SLSDC) contracted with the Corps of Engineers (Corps) to design and construct Eisenhower and Snell Locks. This paper will present a historical overview of Corps involvement over the past 36 years in support of the SLSDC mission related to lock construction and maintenance.

Buffalo District designed and constructed Eisenhower and Snell Locks between 1956 and 1958. Soon after the first operating season in 1958, construction-related structural problems and/or deficiencies began to appear. A 2-year major rehabilitation program to restore Eisenhower and Snell Locks to a condition of full stability was completed by Buffalo District during 1969. Throughout the years the Corps has provided technical advice to SLSDC to assist their efforts to cope with the extraordinary maintenance associated with these locks.

Buffalo District reviewed the stability of the Eisenhower and Snell Lock walls during 1984-85 and recommended a major rehabilitation to bring the lock walls into compliance with current overturning and sliding criteria. SLSDC engaged a consulting engineer to design the necessary structural modifications. Preliminary results of a Corps-sponsored REMR research program to evaluate the accuracy of conventional stability analysis methods were reported in April 1987 and indicated that the current methods may be too conservative. Based on these results the Corps determined that the lock walls would be stable under the newly developed criteria.

During fall 1988, the Corps proposed a study to evaluate the internal structural integrity of Eisenhower and Snell Lock walls. The study, consisting of field investigations and complex analytical and seismic analyses, is currently under way at WES. This paper will be coauthored by Buffalo District and WES in order to present an accurate discussion of the current study effort.

The following topics will be discussed in this presentation:

- *Postconstruction structural problems.*
- *1968-69 major rehabilitation program.*
- *Record of structural repairs over the years.*

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- *Reanalysis of lock wall stability during changing times.*
- *1989 anchor investigation study and conclusions.*
- *Current structural evaluation and preliminary results.*
- *Seaway impact on Corps policy over the years.*

Structural Reliability and Its Impact on Design

by
Nathan M. Kathir, PE[1]

Abstract

The American Institute of Steel Construction (AISC) issued the first edition of a code based on load and resistance factor design (LRFD) in 1986. Before the LRFD was published in the US, structural steel design has been done by using the allowable stress design (ASD) method. At the present time, other code writing authorities in the US are exploring the possibility of issuing LRFD-based codes. Keeping up with the latest developments in the field is an integral part of total design quality. The primary purpose of this paper is to introduce the design engineer to the concept of structural reliability and its potential influence on structural design. This paper reviews existing design methods and traces the development of LRFD format. Basic structural reliability theory is presented, and its relationship to LRFD is explained. Advantages and disadvantages in using LRFD-based codes are also discussed.

Introduction

In a large organization such as the Corps of Engineers, the end quality of a product depends on the combined effort of many professionals having very diversified fields of expertise. Management, quality control, design, peer review, value engineering, construction management, etc. contribute toward producing this quality product to meet a customer's needs. This paper will emphasize the theme of the conference, "Total Design Quality." An engineer, even after attaining professional status by obtaining the license of a Professional Engineer, must continually educate himself/herself to remain technically proficient and current. Keeping up with the latest developments in the field is an integral part of total design quality.

The method of structural steel design in the US has undergone major changes within the last 5 years. Most of the building codes in the US adopt the specifications by the American Institute of Steel Construction (AISC) for use in structural steel design. At present, AISC has two versions of the code. One version is the 9th Edition (AISC 1989) of the traditional Allowable Stress Design (ASD) code, also known as the working stress method. The other version is the first edition of the Load and Resistance Factor Design (LRFD) code (AISC 1986). An engineer in private practice has the option of using either one of the codes at the present time. However, it is very likely that in the near future only the LRFD code will be in use. The primary purpose of this paper is to review and summarize the theory and rational behind the development of the LRFD code. Having an understanding of the development will certainly benefit an engineer trying to use the code. For the sake of completion, the paper will start with a review of the common design philosophies in structural design and the methods currently used in some building materials.

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Design Philosophies

There are two philosophies of structural design in current use (Salmon and Johnson 1990). One is the working stress method of design (WSD), also referred to as the ASD. AISC, the American Association of State Highways and Transportation Officials (AASHTO), the timber industry, etc. have used the ASD for many years. In ASD, design is done corresponding to the actual 'working load' and the calculated 'working' stresses are compared with a set of allowable stresses. The allowable stresses are usually obtained by dividing the failure stresses by a corresponding factor of safety. For example, tensile failure stress in steel corresponds to its yield stress.

The second philosophy of design is the limit states design (LSD). The methods such as strength design method (SDM), load factor design, ultimate strength design (USD), plastic design, and the load and resistance factor design are all some form of LSD. In this philosophy of design, a structure and its elements are checked at various "limit" states. A *limit* state is a condition in which a structure or one of its component ceases to provide its intended function. A limit state does not necessarily imply a physical failure in the structure. There are two types of limit states. At the serviceability limit state, a structure becomes unable to provide its intended function. At the strength limit state, the structure becomes unsafe. In an LRFD approach, strength limit states are checked because of the concern for safety. However, in some situations, serviceability limit states such as deflection, vibration, crack width (in concrete structures), etc. are checked after a design is done using the strength limit state. In the SDM, design loads are multiplied by a "load" factor, and the resulting stresses are compared with the corresponding "failure" strength. In the SDM, a designer gets a feel" for the structure and its members at their "failure" state and is able to analyze the structure more rationally.

Within the Corps of Engineers, the Engineer Manuals and Engineer Technical Letters are some of the guides available to designers. EM 1110-1-2101, "Working Stresses for Structural Design" (Headquarters, Department of the Army, 1972) gives the allowable stresses for various materials under different conditions. ETL 1110-2-312, "Strength Design Criteria for Reinforced Concrete Hydraulic Structures," (Headquarters, Department of the Army, 1988) is used for reinforced concrete hydraulic structures with the SDM. The load factors given in the ETL 1110-2-312 are higher than those given in the American Concrete Institute (ACI) 318-89 code (ACI 1989). This ETL also specifies the use of 48-ksi yield strength for Grade 60 steel. At the present time, a draft version of an Engineer Manual, EM 1110-2-XXXX, "Strength Design for Reinforced Concrete Hydraulic Structures" (Headquarters, Department of the Army, draft version), is available. This EM specifies the use of Grade 60 steel with a yield strength of 60 ksi and load factors which are the same as those specified in the ACI code except for hydraulic structures. For hydraulic structures, the load factors are multiplied by a factor of 1.30.

Reinforced concrete design

In reinforced concrete design, "Building Code Requirements for Reinforced Concrete," ACI 318-89, (ACI 1989) is adopted by many code authorities. Until 1963, ACI had used only the WSD in its code. In the 1963 edition, it introduced an alternate method of design known as USD. As we are all familiar now, the present ACI code has the SDM as the primary method with the WSD as an alternate design method. Although WSD is still used in a few special circumstances, SDM is the widely used design method for reinforced concrete design. Work is presently underway to reevaluate the load and resistance factors based on reliability analysis for the ACI code.

Other structural designs

At the present time, the wood industry is developing an LRFD specification for wood construction. AASHTO specifies the use of one form of load factor design and is in the process of developing an LRFD based code.

The American Petroleum Institute has explored the possibility of adopting an LRFD based code for the design of offshore structures. With the above mentioned developments, a structural engineer is bound to come across designs done using an LRFD specification. In the undergraduate civil engineering curriculum, many universities across the country have started teaching the LRFD based design. Therefore, a structural engineer must get comfortable with, understand, and be able to use LRFD based specifications.

Structural Reliability

Other branches of engineering have used the theory of reliability for a long time. In structural engineering, the theory of reliability has been utilized primarily during the last 2 decades. In an engineering design, the mere fact we use a safety factor is our tacit acceptance that some of the assumptions in the analysis, design, and construction are not known with complete accuracy. A safety factor is used even when an analysis is correct which indicates that material strengths (resistance) and the design loads are not known with enough accuracy. In fact, it is almost impossible to accurately predict some of the future live loads. There is always a possibility for understrength of the material or for overloading to occur. Therefore, it becomes logical to treat the design variables statistically. Structural reliability theory is a tool to treat those parameters as random variables. Safety can be assured only in terms of probability that the available material strength will be able to withstand the possible maximum load during the design life.

Consider a structural member with a nominal resistance of R, subjected to a nominal load effect of Q. Note that, in general, both R and Q are random variables. Safety margin M of the member is

$$M = R - Q \tag{1}$$

Expression for the safety margin M is also known as failure function. Whenever Q exceeds R, the member is considered failed. The probability of failure is

$$P_f = P(M < 0) \tag{2}$$

The random variables R and Q can have any type of distributions. Also, note that Equation 1 represents a simple form of the safety margin. It is quite possible that the load effect term Q could be a combination of multiple loads and the expression for safety margin could get more complicated. However, for the sake of explanation, the discussion is continued with the expression for M as shown in Equation 1.

Probability density (frequency) functions for the load and the resistance variables together with the terminology used are shown in Figure 1. When the density functions overlap as shown, then there is a possibility of failure. Probability of failure can be qualitatively indicated within the overlap region as shown in Figure 1. Probability of failure is given by a convolution integral. When the random variables R and Q are statistically independent,

$$P_f = \int_{-\infty}^{\infty} F_R(x)\, f_Q(x)\, dx \tag{3}$$

or

$$P_f = \int_{-\infty}^{\infty} [1 - F_Q(x)]\, f_R(x)\, dx \tag{4}$$

Probabilities of failure and success (nonfailure) are always complimentary and therefore,

$$P_s = 1 - P_f \tag{5}$$

As it can be noticed from Figure 1, probability of failure depends on the relative positions as well as the degree of dispersion of the density functions.

Concept of Reliability Index

Consider a case where both R and Q are normally distributed and statistically independent. A normally distributed random variable R which has a mean of μ and a standard

deviation σ is denoted by $N(\mu,\sigma)$. Therefore, R and Q may be written as:

$$R = N(\mu_R, \sigma_R) \quad (6)$$

$$Q = N(\mu_Q, \sigma_Q) \quad (7)$$

Since R and Q are normally distributed, the variable M is also normally distributed. Let

$$M = N(\mu_M, \sigma_M) \quad (8)$$

The random variable M can be transformed into a standard (unit) normalized variable Z using the transformation

$$Z = \frac{(M - \mu_M)}{\sigma_M} \quad (9)$$

A standard normal distribution Z can be denoted by N(0,1). Using the definitions given in Equations 2 and 9, the probability of failure becomes as

$$P_f = P\left(Z \le \frac{-\mu_M}{\sigma_M}\right) \quad (10)$$

That is

$$P_f = \phi\left(\frac{-\mu_M}{\sigma_M}\right) = \phi(-\beta) \quad (11)$$

where ϕ is standard normal distribution function and

$$\beta = \frac{\mu_M}{\sigma_M} = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad (12)$$

In Equation 11, μ_M/σ_M is replaced by β, defined as the reliability index. Therefore,

$$\mu_M = \beta\,\sigma_M \quad (13)$$

From Figure 2 and Equation 13, it can be noticed that the mean margin of safety μ_M is at a distance of β times σ_M from the origin. Knowing β, the probability of failure can be obtained from the distribution of a standard normal curve.

The preceding discussion involved the definition of reliability index for a structural member or element having a simple linear failure function involving two normally distributed, statistically independent variables. There have been methods established in the literature for defining reliability involving nonnormal variables, correlated variables, and complex failure functions. Three such references are Hasofer and Lind (1974), Ellingwood, et al. (1980), and Thoft-Christenson and Baker (1982). For log-normally distributed R and Q, it has been shown that the reliability index is approximately equal to

$$\beta = \frac{\ln\left[\frac{\mu_R}{\mu_Q}\right]}{\sqrt{V_R^2 + V_Q^2}} \quad (14)$$

where V_R and V_Q are the coefficients of variation of the variables R and Q, respectively.

Probability Based Design

Previous discussion involved the definitions of reliability and the reliability index β. To be of practical use, the β must be taken into consideration in design. To obtain the design factors, denominator of Equation 14 is approximated as (Ravindra and Galambos (1978), and Pinkham and Hansell (1978))

$$\sqrt{V_R^2 + V_Q^2} \cong \alpha(V_R + V_Q) \quad (15)$$

where α is a constant. With the approximation of Equation 15, Equation 14 reduces to

$$\mu_R \exp(-\alpha\beta V_R) = \mu_Q \exp(-\alpha\beta V_Q) \quad (16)$$

Defining

$$\phi_1 = \exp(-\alpha\beta V_R) \quad (17)$$

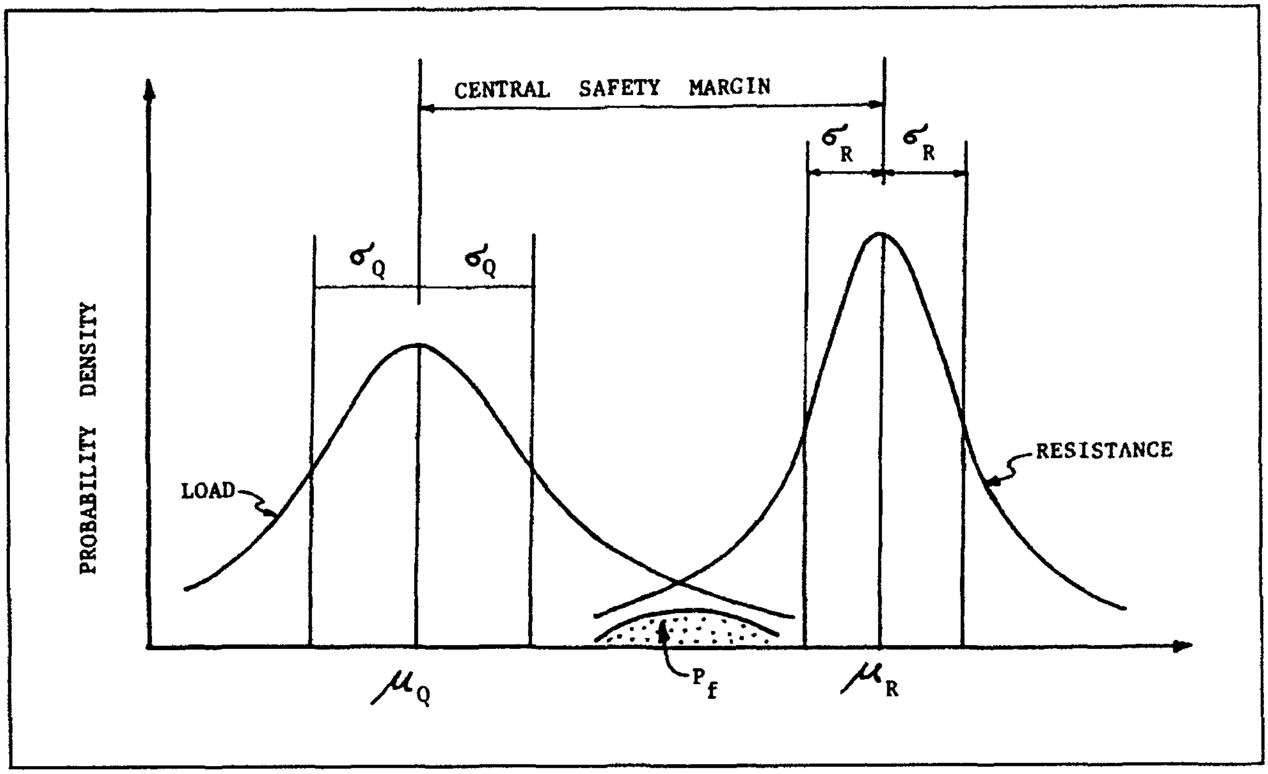

Figure 1. Load and resistance distributions

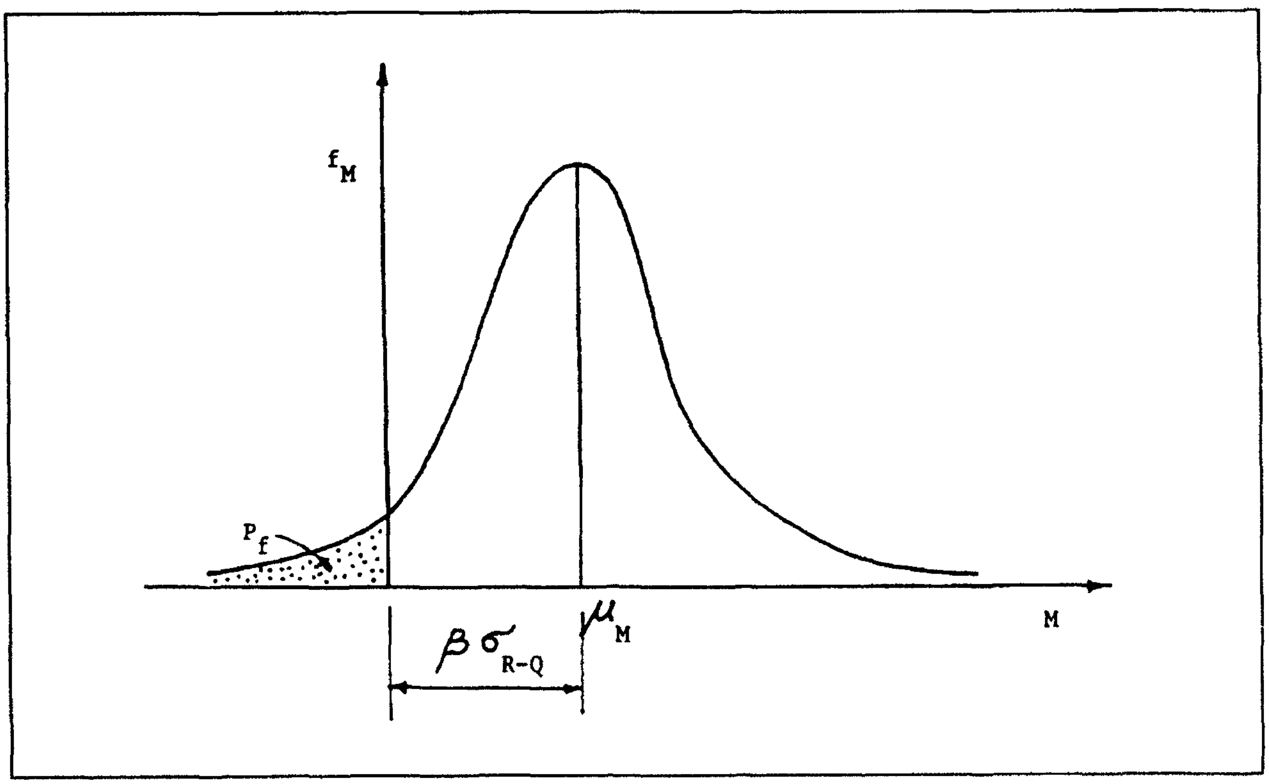

Figure 2. Reliability index concept

$$\gamma_1 = \exp(-\alpha\beta V_Q) \tag{18}$$

Equation 16 becomes

$$\phi_1\mu_R = \gamma_1\mu_Q \tag{19}$$

A bias factor b is defined as the mean value divided the characteristic or nominal value of a random variable. In terms of the nominal values, Equation 19 is written as

$$\phi_1 b_R R_n = \gamma_1 b_Q Q_n \tag{20}$$

Equation 20 is written in the familiar design form as:

$$\phi R_n \geq \gamma Q_n \tag{21}$$

Note that Equation 21 is the design format used in the LRFD design.

$$\phi = \text{resistance factor} = f(\beta, V_R) \tag{22}$$

$$\gamma = \text{load factor} = g(\beta, V_Q) \tag{23}$$

Each load and resistance factor depends only on its uncertainty (i.e., the scatter of the variable V) and the reliability index β. In other words, load and resistance factors are chosen to reflect a certain level of "predetermined" safety through the use of reliability index. Code writing authorities decide the required level of safety considering other factors such as construction practices, and environmental, social, and economic considerations. In the format available to a designer, the LRFD format as shown in Equation 21 is simple to use as previously familiar ASD methods.

The load and resistance factors given in the AISC LRFD Specifications (AISC 1986) were obtained after extensive research and discussion. Research was done to obtain material and load characteristics and to calibrate the load and resistance factors with ASD specifications. The factors given in the LRFD specifications are supposed to give uniform reliability for all members. Note that in Equation 21 the right-hand side could include more than one term corresponding to the load effects of multiple loads acting concurrently. In that case, each load would have independent load factor. Note that the discussion heretofore has dealt only with member reliability. Another area where research is still being done is the system reliability. It is conceivable that the load and resistance factors may be modified in the future to account for advantages arising from structural system reliability.

Benefits of LRFD

One question that needs an answer is "Why LRFD?" It is another way of proportioning structural members. When the LSD was introduced in reinforced concrete design, the load factors were introduced based on the previous design experience and on an elementary statistical model rather than an extensive probabilistical analysis (MacGregor 1988). In effect, in an LRFD based design, the structural members are designed to reflect a predetermined level of safety. Structural reliability theory has helped the code writers to quantify the risk involved.

Saving in materials is also possible using LRFD. In cases where the ratio of live load to dead load (L/D) decreases below unity, the LRFD format gives lighter members compared to that obtained using ASD. On the other hand, when the ratio L/D increases above unity, LRFD may not produce savings in materials. However, it assures uniform reliability. In a study by AISC (1988), it was reported that in office buildings, designs using LRFD resulted in average weight savings of 6.6 percent for beams and 3.6 percent for columns for a combined average savings of 5.5 percent. In parking structures, combined average savings was 10.1 percent. Composite floor systems produced a savings of 4 to 12 percent. Although the percentage of saving using LRFD may not be very high, the method provides a uniform level of safety for all members.

Many advantages of the LRFD method were explained in detail by Beedle (1986) and

are summarized below. LRFD is another design tool for the structural engineer and an added option. ASD is an approximate way to account for what LRFD does in a more rational way. Use of individual load factors will lead to savings in materials. The LRFD makes the design in all materials more compatible and gives the designer a framework needed to handle any unusual situation. The LRFD method also accommodates the input of new information on loads and load variations when that information becomes available. Similarly, new information on materials also can be easily incorporated. Finally, the LRFD makes the design in all materials more compatible.

LRFD and the Corps

The last item that must be addressed is "Should the Corps of Engineers adopt the LRFD format?" The decision authority lies with Headquarters, US Army Corps of Engineers. It is the author's opinion that the structural engineers in the Corps should be familiar with the method and could have it as an option. The Corps can adopt the LRFD method and still retain the desired level of reliability through the use of appropriately calibrated load and resistance factors. Once guidance is available on LRFD, the routine use of the method in design should not pose any difficulty for structural engineers. An added benefit of understanding the reliability theory is that it can be utilized in other areas where a probabilistic risk assessment becomes necessary or useful.

References

American Concrete Institute. 1989. *Building Code Requirements for Reinforced Concrete, ACI 318-89, and Commentary, ACI 318R-89*, Detroit, MI.

American Institute of Steel Construction. 1986. *Manual of Steel Construction - Load and Resistance Factor Design*, Chicago, IL.

________. 1988. Lecture Notes on *Economy in Steel, ASD versus LRFD*, Chicago, IL.

________. 1989. *Manual of Steel Construction - Allowable Stress Design*, Chicago, IL.

Beedle, L.S. 1986. "Why LRFD," *Modern Steel Construction*, American Institute of Steel Construction, Fourth Quarter, pp 30-31.

Ellingwood, B., Galambos, T. V., MacGregor, J. G., and Cornell, C. A. 1980. *Development of a Probability Based Load Criterion for American National Standard A58*, NBS Special Publication 577, US Dept. of Commerce, Washington, DC.

Hasofer, A. M., and Lind, N. C. 1974. "Exact and Invariant Second-Moment Code Format," *Journal of Engineering Mechanics Division*, American Society of Civil Engineering, Vol 100, No. EM1, pp 111-121.

MacGregor, J. G. 1988. *Reinforced Concrete: Mechanics and Design*, Prentice Hall, Englewood Cliffs, NJ.

Pinkham C. W., and Hansell, W. C. 1978. "An Introduction to Load and Resistance Factor Design for Steel Buildings," *Engineering Journal*, American Institute of Steel Construction, First Quarter, pp 2-7.

Ravindra, M. K., and Galambos, T. V. 1978. "Load and Resistance Factor Design for Steel," *Journal of Structural Division*, American Society of Civil Engineering, Vol 104, No. ST9, pp 1337-1353.

Salmon, C. G., and Johnson, J. E. 1990. *Steel Structures - Design and Behavior*, Harper Collins Publishers, New York, NY.

Thoft-Christenson, P., and Baker, M. J. 1982. *Structural Reliability Theory and Its Applications*, Springer-Verlag, Berlin, Germany.

Headquarters, Department of the Army. 1972. *Working Stresses for Structural Design*, Engineer Manual 1110-1-2101, Washington, DC.

US Army Corps of Engineers 1988. *Strength Design Criteria for Reinforced Concrete Hydraulic Structures*, Engineer Technical Letter 1110-2-312, Washington, DC

________. 1990. *Strength Design for Reinforced Concrete Hydraulic Structures*, Engineer Manual 1110-2-XXXX (Draft Version), Washington, DC

Lateral Stability of Beams Loaded by Transverse Members Bearing on Their Top Flanges

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Abstract

The moment capacity of steel beams is often limited by stability considerations. Lateral restraint from diaphram action inhibits lateral torsional buckling in beams supporting cast-in-place decks. Similar lateral restraint is hard to provide, however, when other decking systems such as timber or precast planks are used.

The closed-form solutions presented here take into account the restoring moments exerted by transverse members on the top flange of the beam on which they bear and shows that lateral torsional buckling is greatly inhibited by these restoring moments.

Introduction

There have been many treatments of the problem of lateral torsional buckling of beams. A common approach has been to modify the solution for the beam subject to pure bending under constant moment with multipliers as is done by the American Institute of Steel Construction in the *AISC Manual of Steel Construction* (The C_b coefficients). While this approach adequately takes into account moment gradient, it ignores the manor in which the beam is actually loaded. In practice, beams are almost never loaded by end couples alone. Moments are produced by transverse loads. How those loads are applied can greatly affect the lateral stability of the beam.

Development of Solution

Figure 1 shows the undeflected and deflected state of a beam undergoing lateral torsional buckling. Note the term $C*\phi/W_{(s)}$. This represents the tendency for the resultant location of the load $W_{(s)}$ to move with respect to the twist angle ϕ. The constant C relates the restoring moment produced by this eccentricity to the twist angle ϕ and is a function of the stiffness of the top flange and the member bearing upon it. The solution will proceed by defining the total energy of a variation in terms of the local coordinate system, U, V, ϕ, and setting that energy equal to zero.

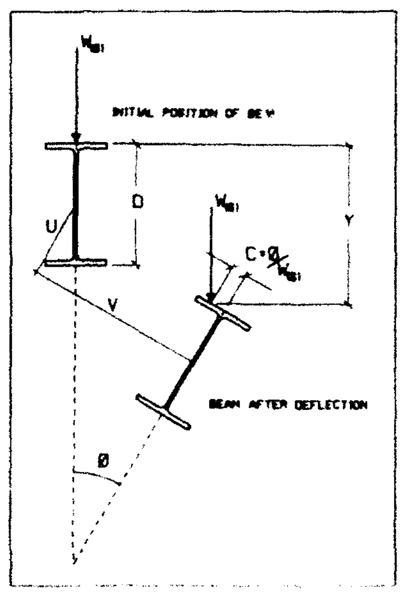

Figure 1. Lateral torsional buckling

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Total internal strain energy Φ is given by:

$$\int_0^L \frac{1}{2} * \left\{ E * \left[I_{xx} * (U''^2) + I_{yy} * (V''^2) + \Gamma * (\phi''^2) \right] + G * J * (\phi'^2) \right\} ds \tag{1}$$

where

I_{xx} = major axis moment of inertia

Iyy = minor axis moment of inertia

G = shear modulus

J = torsional constant for the section

Γ = warping constant for the section

s = distance along the beams longitudinal axis

Primed terms are derivatives with respect to s .

The total external potential energy, Ω, due to the applied load $W_{(s)}$ is given by:

$$\int_0^L \left\{ W_{(s)} * \left[V * \sin(\phi) + U * \cos(\phi) - \left(\frac{C * \phi}{W_{(s)}} \right) * \sin(\phi) + \frac{D}{2} * (1 - \cos(\phi)) \right] \right\} ds \tag{2}$$

where $W_{(s)}$ = load function of s .

Consider the special case in which $W_{(s)}$ is simply a point load at $s = L/2$ caused by a transverse member bearing on a simply supported beam, and assume that U , V , and ϕ are given by the following:

$$U = A_1 * \sin\left(\frac{\pi * s}{L}\right)$$

$$V = A_2 * \sin\left(\frac{\pi * s}{L}\right)$$

$$\phi = A_3 * \sin\left(\frac{\pi * s}{L}\right)$$

Substituting these three expressions for U , V , and ϕ into Equations 1 and 2 yields the following:

$$\Phi = \frac{\pi^4}{4L^3} * [E * I_{xx} * A_1^2 + E * I_{yy} * A_2^2 + (E * \Gamma + G * J) * A_3^2] \tag{3}$$

$$\Omega = -P \left\{ A_2 * \sin(A_3) + A_1 * \cos(A_3) - \frac{C}{P} * A_3 * \sin(A_3) + \frac{D}{2} * [1 - \cos(A_3)] \right\} \tag{4}$$

The total energy of a variation with respect to the $A_i s$ in these equations must be zero. This implies that:

$$\frac{d\Phi}{dA_i} + \frac{d\Omega}{dA_i} = 0 \qquad \text{for all } A_i \tag{5}$$

At the onset of instability, A_3 will be infinitesimal, therefore $\sin(A_3) \approx A3$ and $\cos(A_3) \approx 1$. Using these approximations and inserting Equations 3 and 4 into Equation 5 for all i yields the following set of three simultaneous equations:

$$\text{for } i = 1 \longrightarrow A_1 = \frac{2*P*L^3}{\pi^4*E*I_{xx}} \tag{6}$$

$$\text{for } i = 2 \longrightarrow A_2 = \frac{2*P*A_3*L^3}{\pi^4*E*I_{yy}} \tag{7}$$

$$\text{for } i = 3 \longrightarrow \left[\frac{G*J*\pi^2}{2*L} + \frac{E*\Gamma*\pi^4}{2*L^3}\right]*A_3 = P*\left[A_2 - A_3*A_1 + \left(\frac{D}{2} - 2*\frac{C}{P}\right)*A_3\right] \tag{8}$$

Combining Equations 6, 7, and 8 yields:

$$P^2*\left[\frac{2*(I_{xx} - I_{yy})*L^3}{E*I_{xx}*I_{yy}*\pi^4}\right] + P*\frac{D}{2} - \left(\frac{\pi^2}{2*L}\right)*\left[G*J + \frac{\pi^2}{L^2}*E*\Gamma + \frac{4*C*L}{\pi^2}\right] = 0 \tag{9}$$

Solving for P using the quadratic formula yields:

$$P = \frac{-\frac{D}{2} + \sqrt{\left[\frac{D}{2}\right]^2 + 4*\left[\frac{(I_{xx} - I_{yy})*(G*J*\pi^2*L^2 + \pi^4*E*\Gamma + 4*C*L^3)}{E*I_{xx}*I_{yy}*\pi^4}\right]}}{\left[\frac{4*(I_{xx} - I_{yy})*L^3}{E*I_{xx}*I_{yy}*\pi^4}\right]} \tag{10}$$

A similar solution can be developed for the case of a simply supported beam under a uniform load, W. Equation 2 becomes:

$$W*\int_0^L \left\{A_2 * \sin\left(\frac{\pi*s}{L}\right) * \sin\left[A_3 * \sin\left(\frac{\pi*s}{L}\right)\right] + A_1 * \sin\left(\frac{\pi*s}{L}\right) * \cos\left[A_3 * \sin\left(\frac{\pi*S}{L}\right)\right]\right.$$

$$\left. - \left(\frac{C*A_3}{W}\right) * \sin\left(\frac{\pi*s}{L}\right) * \sin\left[A_3 * \sin\left(\frac{\pi*s}{L}\right)\right] + \frac{D}{2} * \left\{1 - \cos\left[A_3 * \sin\left(\frac{\pi*s}{L}\right)\right]\right\}\right\} ds$$

The differentiating with respect to A_i is best carried out before the integration with respect to s is performed. If this is done and use is made of the following approximations:

$$\sin\left[A_3 * \sin\left(\frac{\pi*s}{L}\right)\right] \approx A_3 * \sin\left(\frac{\pi*s}{L}\right) \quad , \quad \cos\left[A_3 * \sin\left(\frac{\pi*s}{L}\right)\right] \approx 1$$

then Equations 6, 7, and 8 become:

$$\text{for } i = 1 \longrightarrow A_1 = \frac{4*W*L^4}{\pi^5*E*I_{xx}}$$

$$\text{for } i = 2 \longrightarrow A_2 = \frac{W*A_3*L^4}{\pi^4*E*I_{yy}}$$

$$\text{for } i = 3 \longrightarrow \left[\frac{G*J*\pi^2}{2*L} + \frac{E*\Gamma*\pi^4}{2*L^3}\right]*A_3$$

$$= W*\left\{\left[A_2 + \left(\frac{D}{2} - 2*\frac{C}{W}\right)*A_3\right]*\left(\frac{L}{2}\right) - A_3*A_1*\left(\frac{4*L}{3*\pi}\right)\right\}$$

As with the case of the point load, when the above equations are combined, the A_i terms vanish, leaving the following expression for the critical uniform load, W :

$$W = \frac{-\frac{D}{2} + \sqrt{\left[\frac{D}{2}\right]^2 + \left[\frac{(3*\pi^2*I_{xx} - 32*I_{yy})*(G*J*\pi^2*L^2 + \pi^4*E*\Gamma + 2*C*L^4)*L^2}{3*E*I_{xx}*I_{yy}*\pi^6}\right]}}{\left[\frac{(3*\pi^2*I_{xx} - 32*I_{yy})*L^5}{3*E*I_{xx}*I_{yy}*\pi^6}\right]} \tag{11}$$

In both Equations 10 and 11, the factor C augments the torsional resistance of the beam having the effect of inhibiting lateral torsional buckling. The significance of this virtual restraint increases with beam length and eventually dominates the torsional and warping restraint. For large values of L, Equation 11 approaches:

$$W = \frac{1}{L_2} * \sqrt{\frac{6*C*E*I_{xx}*I_{yy}*\pi^6}{3*\pi^2*I_{xx} - 32*I_{yy}}} \tag{12}$$

The significance of this virtual restraint can now be seen. The critical uniform load W as limited by lateral torsional buckling is now proportional to $1/L^2$, just as it is when limited by flexural strength. This means that for sufficiently large values of C, lateral torsional buckling will never control the design of a beam regardless of the its length. This fact can be seen in Figure 2. Here the ratio of W, limited by lateral torsional buckling to W limited by beam strength, is plotted with respect to beam length for a particular wide flange section. For $C = 0$, lateral torsional buckling controls for span lengths over 20 ft. For $C \geq 3$, lateral torsional buckling will never control.

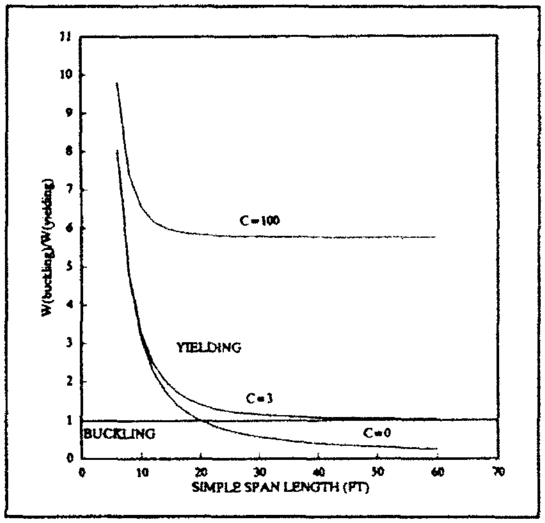

Figure 2. Failure mechanism for W 18 by 55

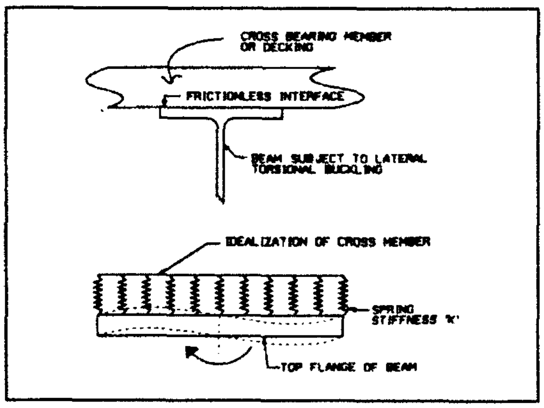

Figure 3. Determination of the constant "C"

The magnitude of C depends on the flexural stiffness of the top flange and the bearing stiffness of the member bearing upon it. Figure 3 shows the top flange of a beam with a cross-bearing member. This situation can be idealized using the Winkler foundation concept. The spring constant "K" of the cross-bearing member of Figure 3 can be conservatively estimated by assuming that the contact stress on the bottom surface of the cross-bearing member decays to zero at the top surface in a linear fashion. This yields the following equation:

$$K = \frac{2*E_d}{H} \tag{13}$$

where H = deck thickness and E_d = deck modulus of elasticity.

From the classical "Beam on a Winkler Foundation" assumption, the value "C" is given by the following:

$$C = 4*E_f*I_f*\lambda* \tanh\left(\frac{\lambda*B_f}{2}\right) \tag{14}$$

where

$\lambda = \sqrt[4]{K/4*E_f*I_f}$

I_f = top flange moment of inertia per beam unit length.

E_f = top flange modulus of elasticity.

B_f = top flange width.

Example

Consider a W 18 by 55 supporting a floor joist system of 2 by 10's at 16 in. on center. Assume the cross grain modulus of elasticity for the deck corrected for joist spacing is:

$$\frac{300*1.5}{16} \longrightarrow E_d = 28.125 \text{ ksi}$$

From Equation 13, K = 5.92 kci . For a W 18 by 55, $I_f = (0.63)^3/12 = 0.0208$ in.^{4}/in. , therefore $\lambda = 0.223$ $B_f = 7.53$ in., so from equation 14:

$$C = 4*29000*0.0208*0.223* \tanh\left(\frac{0.223*7.53}{2}\right)$$

$= 368$ in. kips per inch per radian.

Conclusion

As can be seen from the size of C computed in the previous example and from Figure 2, a wide variety of cross-bearing members or decking should provide sufficient virtual restraint to totally eliminate lateral torsional buckling as a failure mechanism.

Automated Modular Design (Kit-of-Parts) US Army Reserve Center

by
Anjana K. Chudgar, PE[1]

Abstract

The Kit of Parts is a new concept which standardizes functional areas of design while maintaining flexibility in the layout and size of a building. Kit of Parts combines the adaptability of custom design with the speed and consistency of quality.

The Kit of Parts organizes reserve center into basic functional areas; these would include kitchen, administration, drill, storage, maintenance, etc. Several sizes of each area are designed as three-dimensional modules that can be manipulated and arranged to fit specific site and individual customer needs. Alternate configurations of support spaces within modules provide further flexibility. This allows the designer to experiment with various design solutions to suit the diverse needs of the individual customer, and site conditions.

The Kit of Parts consists of computer process quality and computer generated contract document, specifications, schedules, tables, and calculations necessary to create a set of contract documents (architectural, structural, mechanical, plumbing, and electrical) for Reserve Center, exclusive of site-specific requirements. This innovative approach is made possible through the application of our CADD tools and related skills.

US Army Reserve Center in any location in the United States of America can be completely designed by the local Corps District (or its "site AE" firm) and ready to advertise in a matter of weeks.

Introduction

New US Army Reserve Centers (USARC) are treated as separate and unique design with functional relationships and aesthetics that vary considerably. The development of "building blocks" or module for all Reserve buildings reduces administration and design time. These three dimensional standardized modules, while maximizing economy and uniformity, reduce the design to a mere site adapt process taking only a few weeks. In the past, all facilities were designed individually. Standard designs were considered but automated design was considered flexible and economical.

The design concept is based on the development of a number of building plans which increase in size at specified square foot

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increments. One of several architectural styles is then applied to the facade depending on where it is located in the country. This concept eliminates the repetitive design process but does allow necessary flexibility.

The goal for the USARC automated design system modular program is a concept that allows flexibility of layout between functional areas. The module concept is based on program and space relationship analyses to determine what major functional areas in Reserve Centers can be converted into "building blocks" or modules. Even though each block is dependent on the other to a certain degree, each block is a separate identifiable operation (function) within a Reserve Center.

Goal

The Automated Modular Design provides "Total Design Quality." The goals of the modular program for USARC are to:

- Develop a standard plan concept that will allow flexibility between functional areas.
- Maximize user input producing an individually designed product using assemblyline techniques.
- Reduce total design and review time.
- Reduce total design cost.
- Improve quality of construction through standardized plans and specifications and fewer construction modifications.

The cost to develop the Automated Modular Design of USARC is about $2.4 million and approximately 3 years of time. It is estimated that 13 Army Reserve Center projects will pay back the initial investment; after that, every 30 projects built will result in savings of between $4 and $5 million.

A System of Modules

The layout of an Army Reserve Center is easily created by combining modules that cover the basic functional areas, such as Lobby module, Administration module, Eduction module, Unit storage module, Band room module, Rifle range module, Toilet room module, Mechanical room module, etc (Figures 1-3). The sizes of modules are based on square footage requirement provided in "Space Guidelines, US Army Reserve Facilities" (1986). Functionalities, adjacencies and other pertinent data were obtained from "Design Guide for US Army Reserve Facilities." After a through overview of the space requirements for all potential modules, and taking into consideration reasonable structural spans, a 30-ft dimension in at least one direction (constant) and multiples of various bay size in the opposite direction were used as a planning grid.

- Educational modules 30 ft wide by 20 ft deep accommodate two 15 ft wide classrooms which meet AR 140-485 space requirements (300 sq ft each)
- Unit storage modules 30 ft wide by 40 ft deep accommodate 8- by 12-ft wire caging with a 6 aisle between.
- The arms vault 30 ft wide by 24 ft deep (within the Assembly module) also accommodates similar caging and aisle requirements.
- Administration modules work in multiples of 30-ft square bays and can accommodate office and other special training sub-modules located as necessary within this space.

The 30-ft dimension, therefore, should result in an optimum bay size providing necessary flexibility to meet specific program needs. A 15-ft sub-grid is also used to expand spaces as necessary.

The USARC modular program uses conventional building components familiar to contractors throughout the country. A structural system designed for nation-wide use, available on a matrix that provides a designed structural system (by engineer selection) to meet specific local (wind, seismic,etc.) code requirements. With the selective choice of

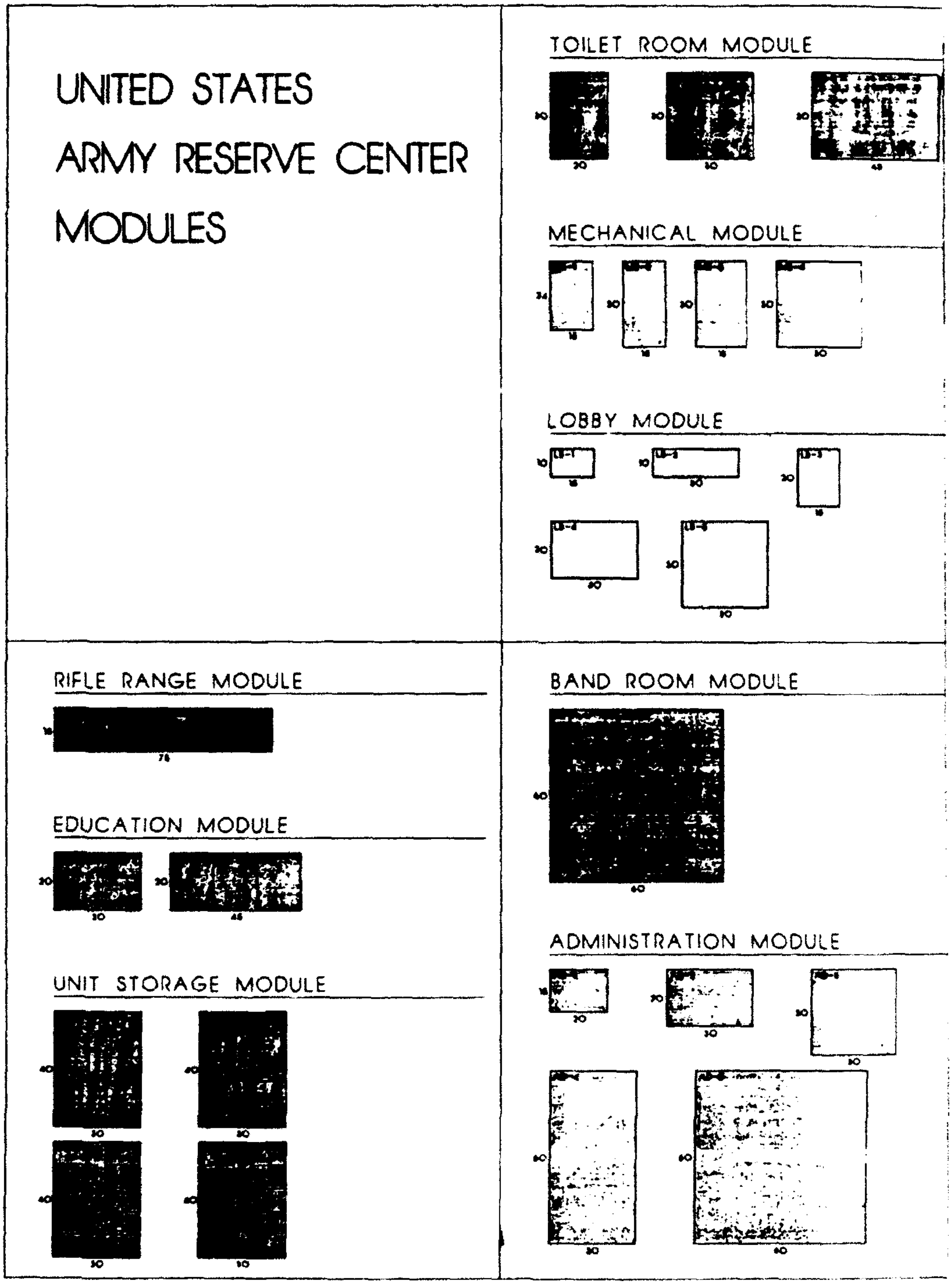

Figure 1. US Army Reserve Center modules

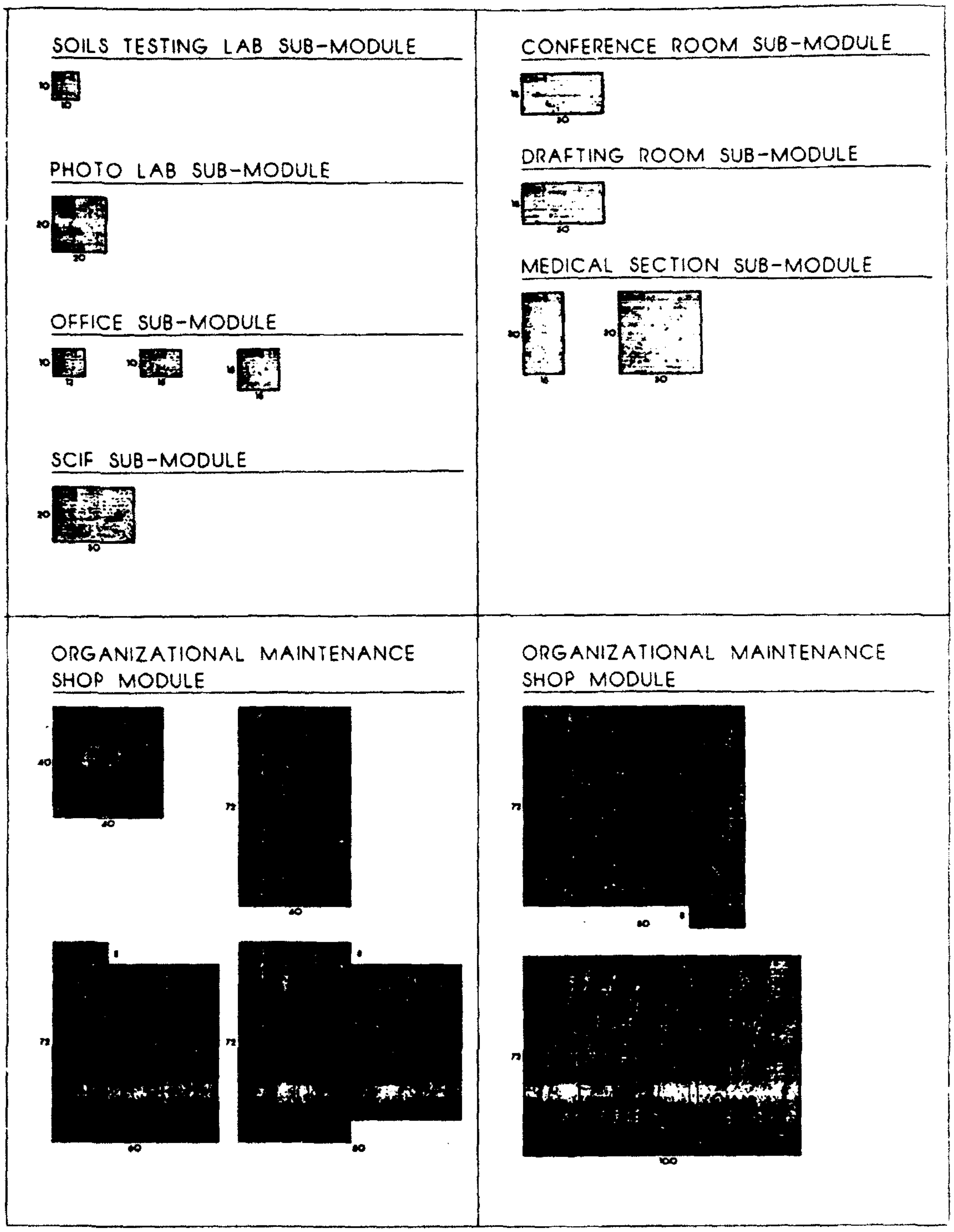

Figure 2. Additional modules, US Army Reserve Center

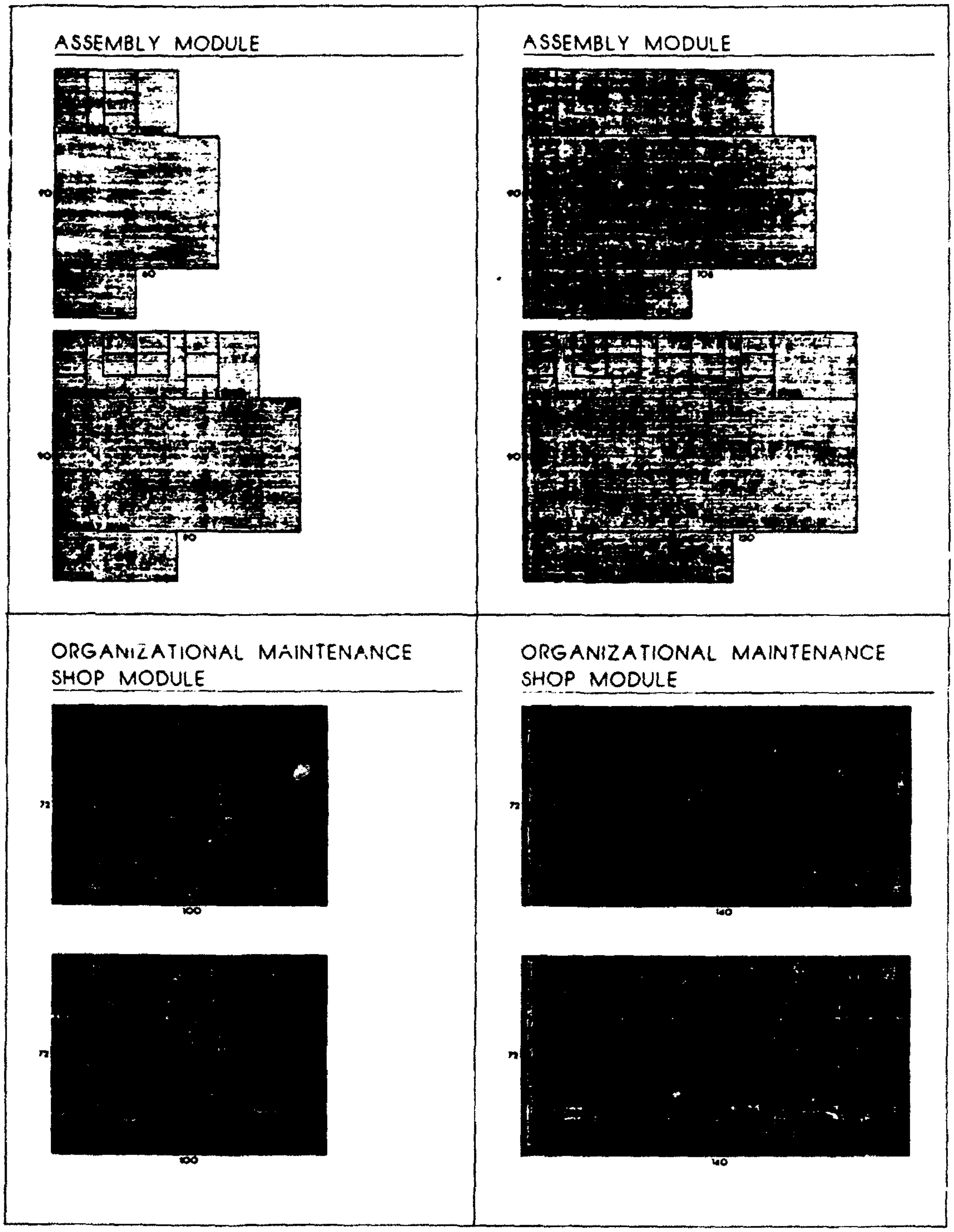

Figure 3. Assembly modules, US Army Reserve Center

subsystems, it becomes a fully integrated network of services appropriate to the spatial needs of the program. The structure uses these components and basic construction skills as a building system. Where possible, all structural members—column, beam, and joist—are identical, fixed in height, length, and size. An efficient and economical framing system must respond to functional needs while accommodating varying wind, snow, and seismic loadings.

Each functional area is represented by many modules, each of a different size. By selecting a module from each required family of parts and arranging them in the proper configuration, one can create Reserve Centers that vary in size and programmatic need. Submodules comprised of general offices as well as special training areas can be placed in different locations within major modules.

The program takes the "site A/E" firm well into the design, leaving only site-specific items to address. This work included all civil, landscape, foundation, and slab design and sizing of mechanical, plumbing, and electrical equipment and systems. This is necessary because of environmental, geotechnical and generally unpredictable design parameter differences across the United States. The basic architecture and graphics of the building should be constant to reinforce the identity of "US Army Reserve Center" throughout the country. Various facade materials included in the modular program should be selected to achieve connection to and harmony with region and locale. Easily maintained materials should be specified for all work areas of the building. Neutral colors with lively accents can create a dynamic, energetic quality while providing a pleasant work environment.

CADD

The modular program consists of computer generated drawings, specifications, schedules, tables, and calculations necessary to create a set of contract documents to construct Reserve Centers exclusive of site-specific design features and requirements. The design includes complete architectural, structural, mechanical, plumbing, and electrical drawings and details. These accommodate the standard modules and their typical combinations, including rotated and/or mirrored orientations. Using scale copies of the modules, the group of designers for the USARC plan the layout of the building over the site plan. When the modules to be used have been chosen and arranged on the site plan, they are assembled by CADD into a total Building Plan, and complete working drawings are produced in as little as three to four weeks versus the eighty weeks it took in the past.

Development of USARC Kit-of-Parts

Defining the scope of work for the development of a modular program is a demanding task. The task force personnel assigned to the program must be innovative, forward thinking, and must have a complete understanding of the factors involved in this type of development. Changes occur frequently due to new ideas, special studies, and reports. The task force personnel must be assigned to the project and have the authority to make decisions during the conferences and review meetings. They will provide the continuity required for a successful project. Factors involved in the full development are:

A/E Personnel - Projector Manager, Engineers, Computer Operator, Writer, etc.

Special Consultants - Lighting, Acoustics, Graphics, Security, etc.

Computer - Translation, Tapes, CADD System, Work Processing.

Travel Time

Reproduction - Manual, Reports, Tapes, Drawings, Photographs, etc.

Special Items - Studies, Presentations, Reports.

Modules - Number, Functional Areas, Size.

Conclusion

Program management using the modular program is greatly simplified. Almost all uncertainties in the project delivery process are eliminated with this approach. A projecting any location could be completely designed by the local Corps District (or its "site AE" firm) and ready to advertise in a matter of weeks. The location, arrangement, and orientation of various modules and the building would be determined with the local user, consistent with program allowances and the flexibilities inherent in the Automated Modular Design. This user input is a key feature of the Kit concept, and the critical difference between it and traditional reviews would be eliminated. The developing district would be expected to maintain the primary design files for the Kit of Parts and to oversee any subsequent improvements on the owner's behalf.

References

"Space Guidelines, US Army Reserve Facilities." 1986 (Mar) update. AR 140-485

"Design Guide for US Army Reserve Facilities," DG 1110-3-107.

DD Form 1391's For 11 USARC Project Identifies for Standard Program.

Historical Design Data (Developed floor plans) from Louisville District.

Minutes of 28 February 1990 USARC Standardization Subcommittee Meeting.

Tentative 5 Year MCAR program provided by FORSCOM March 1990.

Seismic Vulnerability and Upgrading of Nonductile Concrete Frames

by
Pamalee A. Brady[1]

Abstract

Recent earthquake events have shown the vulnerability of older nonductile concrete frames (NDCF). Concrete frame buildings constructed prior to 1971 are particularly vulnerable and generally have performed poorly under dynamic loading and large overstresses. Insufficient lateral ties in the critical members, especially columns, provide inadequate confinement of the concrete. The result is a loss of ductility and potential catastrophic brittle failure. Poor behavior in the inelastic range of response was characteristic of NDCF's in the recent Loma Prieta Earthquake in California.

Many essential and high potential loss facilities on Army and Air Force installations were constructed prior to the introduction of the 1971 ACI Code provisions when ductile detailing was required. Seismic analyses of essential and high-potential-loss facilities at numerous installations on the West Coast of the United States have validated this vulnerability.

Current methods for strengthening these NDCF's are costly and/or require significant disruption of use during renovation. Several new technologies however hold promise for improving the behavior of these systems under lateral load and reducing the cost of upgrading. This could have a significant impact on ensuring mission capability and safety in the many existing structures of this type in the DoD. USACERL is conducting a detailed study of the vulnerability of NDCF's in the Army and Air Force inventory. Experimental test specimens of beam/column and flat plate/column subassemblages are being tested to identify their specific response to dynamic loading. Viscoelastic dampers as well as other new technologies are being tested to evaluate the improved frame response using these retrofits techniques. The result of this study will be state-of-the-art design guidance for upgrading the existing essential and high-potential-loss NDCF buildings in the DoD inventory.

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Experimental Testing Of Base Isolator Components

by
James B. Gambill[1] *and Pamalee A. Brady*[1]

Abstract

The US Army Construction Engineering Research Laboratory (USACERL) has begun an experimental test program to evaluate the performance of base isolation components used to protect buildings from the effects of seismic ground motions.

In the past, the approach taken in upgrading existing buildings to resist the effects of seismic ground motion has been to increase the strength or stiffness of the structure, which is usually extremely costly and still poses risks to building contents. In recent years, several manufacturers have developed base isolation systems that protect an entire structure, without major changes to the structure. Most of the experimental verification of base isolators has been vendor-sponsored, and there has been little independent testing to verify the manufacturer's data and to provide engineers and designers with comparative design data.

USACERL has designed a large-scale test facility for performing three-dimensional static, cyclic, and seismic tests of individual base isolator bearings by applying controlled displacements in two horizontal directions, and a controlled load in the vertical axis to simulate the column load of the structure.

A number of isolator manufacturers have agreed to provide samples for comparative testing. The test results will be used to develop design and selection guidance for applying base isolation technology in Department of Defense (DOD), other government agencies, and the public sector.

Introduction

Recent history clearly demonstrates the need to consider seismic hazards when designing or upgrading buildings in regions of seismic activity. To mitigate the effects of earthquakes on a building, the designer attempts to equate a building's capacity to resist earthquake motion (SUPPLY) with the demands placed on the structure by the motion (DEMAND). SUPPLY must be greater than DEMAND.

Traditionally, earthquake safety has been achieved by working on the supply side of the equation either by making the structure strong and stiff enough to resist earthquake motion, or ductile enough to absorb any ground motion applied to it by an earthquake. Both ways allow a building to survive earthquakes without total collapse, but stiff structures can transmit significant vibrations to building contents, causing potential damage; and ductile structures may become unserviceable after the earthquake, requiring major repairs to correct

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permanent deformations that may have occurred in the ductile members. Both of these approaches are effective, but usually result in a conservative design, possibly adding significant cost to the structure.

The cost of such designs, particularly for upgrading existing buildings to meet greater seismic demands, has generated the development of new technologies. Base isolation is one technique which reduces the earthquake demand on a structure by providing a flexible mounting that uncouples the building from the ground motion by lengthening the natural period of vibration of the structure and a damper or energy dissipator, which controls the relative deflections between the building and the ground. These two features reduce the force response of the building. The concept of base isolation is not new; it was introduced in the late 19th century, but interest in base isolation systems has grown rapidly in the recent years as more manufacturers develop base isolation systems.

Individual base isolation systems vary in their details, mounting requirements, damping mechanisms, and therefore their performance. There are elastomeric bearing isolators with a number of variations, roller systems, and friction pendulum systems. Analytical and experimental research has been conducted on the majority of the proposed systems by their manufacturers; however, there is no comparative data for evaluating the manufacturer's claims, or for comparing various isolation systems for use in a particular design. Additionally, past testing has been either single or bi-directional; no tri-directional testing has yet been performed. Triaxial performance parameters have been inferred by assuming that characteristics in the orthogonal, horizontal directions are independent or uncoupled. This is probably a valid assumption for an isolator with a horizontal circular cross section, but cannot be assumed for any other shape.

The Corps of Engineers is interested in applying this new technology to mitigate earthquake hazards associated with existing buildings and to provide seismic resistance in new structures. However, it is difficult to identify the appropriate product without comparative data. For this reason USACERL has initiated a research program to develop comparative data on base isolation systems that may be translated into a performance specification, making base isolation more available for use in military construction.

Research Program Objectives

The objective of this USACERL-initiated research program is to perform comparative biaxial and triaxial testing of base isolators to develop design guidance for the selection and use of the technology in practical structural designs. USACERL has designed a large-scale test facility for performing tri-dimensional static and dynamic tests of individual base isolator components. The isolators will be tested by applying controlled displacements in two orthogonal, horizontal directions. A controlled static load will be applied in the vertical axis to simulate the dead weight of building columns/walls that would be placed on the isolator in a real structure. Static loads will be applied in one or two horizontal axes to determine the basic material properties of the isolators and dynamic tests will be conducted using cyclic displacements or actual seismic motion time histories.

The project objectives will be accomplished in a three-phase program. In Phase I, experimental tests will be performed and a data base of comparative results developed. Phase II will focus on developing analytical models that characterize the experimental results of isolator behavior. Phase III will conclude the work by developing design guidelines, specifications, and construction details. This paper discusses the work being conducted in Phase I. The major project tasks are to:

- Design and construct a triaxial base isolator test fixture.
- Develop and perform a detailed testing program to investigate base isolator behavior under various loading environments.

- Reduce and analyze the test data developed in the experimental test program and develop comparative results that will uniformly demonstrate the performance of the various types of isolators.

Base Isolator Test Specimen Design

It was decided early in the project that testing full-size base isolators would not be economically feasible because of the large vertical column loads and large displacements required to produce the normally accepted 100 percent to 200 percent shear strain in a full-size isolator. Before investigating the scale modeling of base isolators and to ensure that the results would have practical application, base isolation systems were designed for typical real-world buildings to determine appropriate isolator dimensions, and load and displacement magnitudes. The DOD has a large inventory of buildings, which are predominately low- to medium-rise; therefore, structures of this scale were chosen for the design examples. A five-story concrete building and a three-story steel frame building were chosen for analysis purposes. Due to its more common use, an elastomeric-bearing system was used in the sample design for the test structure. Base isolator test specimens were then scaled to represent the isolators of these designs.

To realistically simulate the performance of the full-size design, the laws of similitude had to be followed. Table 1 lists the important similitude relationships related to the performance of elastomeric isolators.

Table 1 shows that the stress, strain, and shape factors are kept constant, and the length, buckling load, roll-out load, and axial stiffness are scaled. The shape factor (S) is defined as: (load-bearing area)/(unloaded perimeter), and is maintained constant because it directly affects the axial compression stiffness (k) and the buckling load (P_B), as shown in Equations 1 and 2.

$$P_B = \frac{1.71 * d^3 * G * S * (t_r + t_s)}{h * t_r}$$

$$k = \frac{E * A * (1 + 1.3 A^2)}{t_r}$$

where:

d = length of one side of a square isolator

G = shear modulus

t_r = thickness of one layer of rubber

t_s = thickness of one steel shim plate

h = height between the end-plates

E = Young's modulus

A = plan area

Because of the material and design requirements for most commercially available elastomeric isolators, it would be difficult to maintain all of the similitude relationships for models less than 1/2 scale; therefore, a 1/2-scale case was chosen for developing the design of the test

Table 1
Elastomeric Isolator Similitude Relationships

Parameter	Relationship	Parameter	Relationship
Stress (axial & shear), s	$s_m = s_p$	Buckling Load, P_B	$(P_B)_m = (P_B)_p \cdot (SF)^{-2}$
Stress (axial & shear), e	$e_m = e_p$	Roll-Out Load, P_{RO}	$(P_{RO})_m = (P_{RO})_p \cdot (SF)^{-2}$
Shape Factor, S	$S_m = S_p$	Axial Stiffness, k	$k_m = k_p \cdot (SF)^{-1}$
Length, l	$l_m = l_p \cdot (SF)^{-1}$		

p = Prototype (full size)
m = Model
SF = Scale Factor.

fixture discussed in the next section. The 1/2-scale elastomeric isolator designed for the concrete building will require the loading fixture to produce approximately 200 kips in the vertical direction to simulate the column gravity load and approximately 90 kips in each of the two horizontal directions based on the maximum shear strength of the isolator. The horizontal loading system must be capable of imposing up to 16 in. of displacement at 200 percent shear strain.

Test Fixture, Loading System, and Instrumentation

The testing of base isolators, even at 1/2 scale, will place enormous demands on the loading actuators, the power supplies, and the test reaction frame. The hydraulic actuators must supply large forces over a long displacement range at relatively high velocities. The reaction frame must be strong and stiff enough to resist the applied forces over the full range of motion of the actuators, and the frame must have a fundamental vibration period far enough above the highest expected test frequency to not significantly interact with the test excitations. With these basic assumptions, three alternative design approaches were investigated. Figure 1 schematically shows the three approaches.

Design A was chosen as the approach for the isolator test frame. Design B was removed from consideration because of the difficulty in fabricating a frictionless sliding surface. The decision was made to use existing facilities at USACERL as much as possible to reduce cost. These facilities include existing hydraulic power supplies. Design C was not chosen because its increased demand on the existing hydraulic power supplies would directly affect the maximum excitation frequencies that could be applied to the isolators.

The reaction frame for design A was designed to support the test isolator and the loading actuators, to be rigid enough to minimize frame member distortion under the worst-case loading, and to facilitate response parameter measurement and actuator control under cyclic and seismic triaxial tests. Figure 2 illustrates the test fixture, showing the relative locations of the actuators and the reaction points. Figures 3 and 4 give detailed plans and elevation views of the test fixture. Table 2 lists the required force and displacement capabilities to meet the performance parameters for the 1/2-scale isolator test.

Actuators H-1 and H-2 (Figure 2) are the primary units for applying the horizontal displacements to the isolator in the two perpendicular horizontal axes. Actuator H-3 is used to prevent rotation of the top reaction block about the vertical axis during the bi-direction horizontal deflections. Actuator V-4 is primary in applying the axial vertical compressive force. Actuators V-5 and V-6 prevent rotation of the top reaction block and the specimen around both of the perpendicular horizontal axes during horizontal translations.

All six actuators incorporate swivel connections on each end to permit the actuators to follow the movement of the top of the isolator during the test while minimizing the lateral reaction loads on the actuators. Each of the horizontal actuator swivel connectors is designed to permit a minimum of ± 10-deg rotation in the horizontal plane and ± 5 deg in the vertical plane. The horizontal rotation allows one actuator to follow displacement in the other actuator. The vertical rotation allows the horizontal actuators to accommodate the vertical compression of the isolator under vertical load, and the axial shortening of the isolator during horizontal translation. The swivel connections on the vertical actuators must allow ± 23 deg of rotation in two directions to allow them to follow the horizontal translations of the isolator. Control algorithms must also be developed for the test fixture loading system to coordinate the actions of the six actuators and accomplish the various loading environments.

The overall reaction frame dimensions are approximately 25 ft by 25 ft in plan at the base and 19 ft by 19 ft in plan at the top, and the height is approximately 11 ft 2 in. (Figures 3 and 4). The test fixture will be constructed of

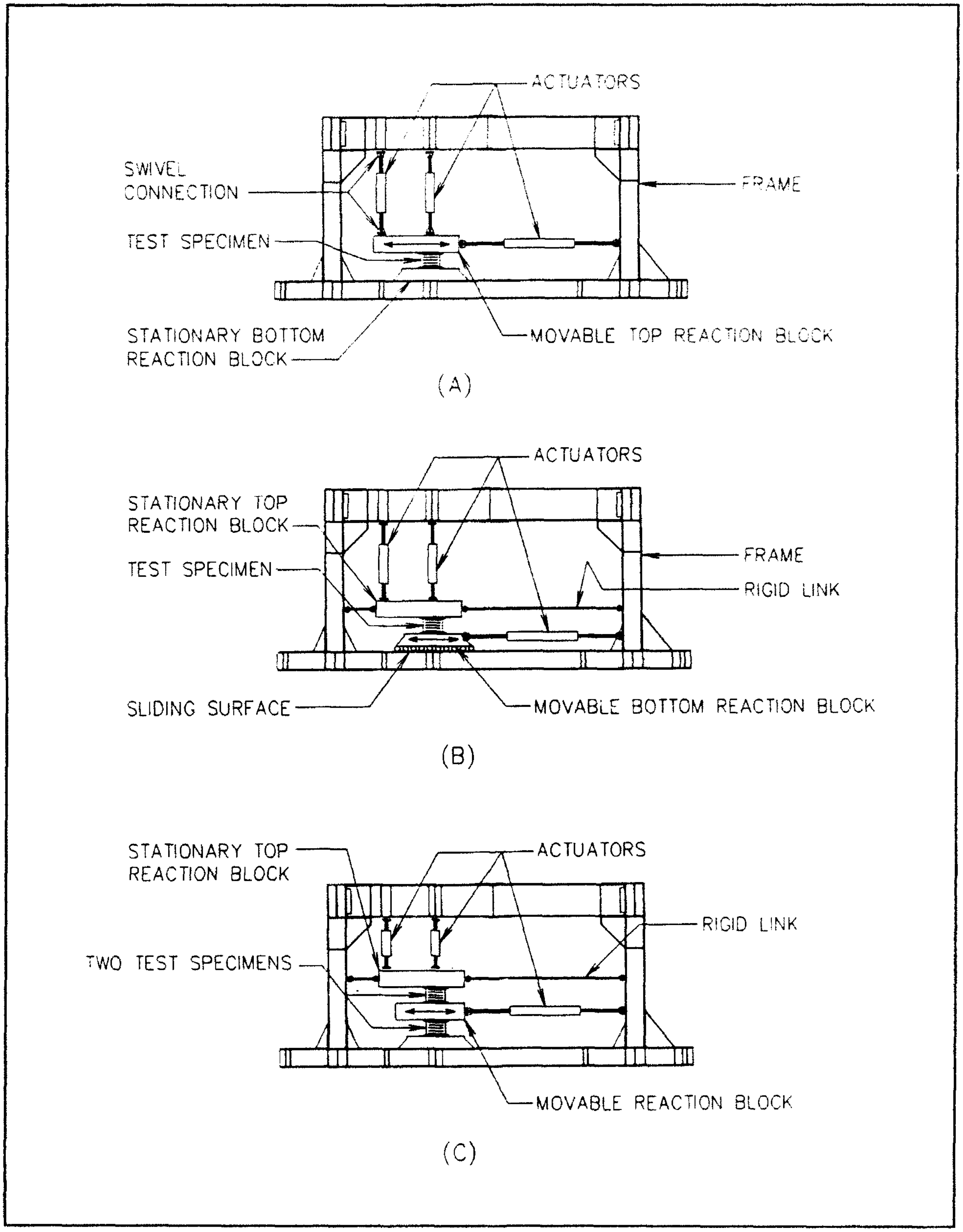

Figure 1. Three design concepts for a test fixture

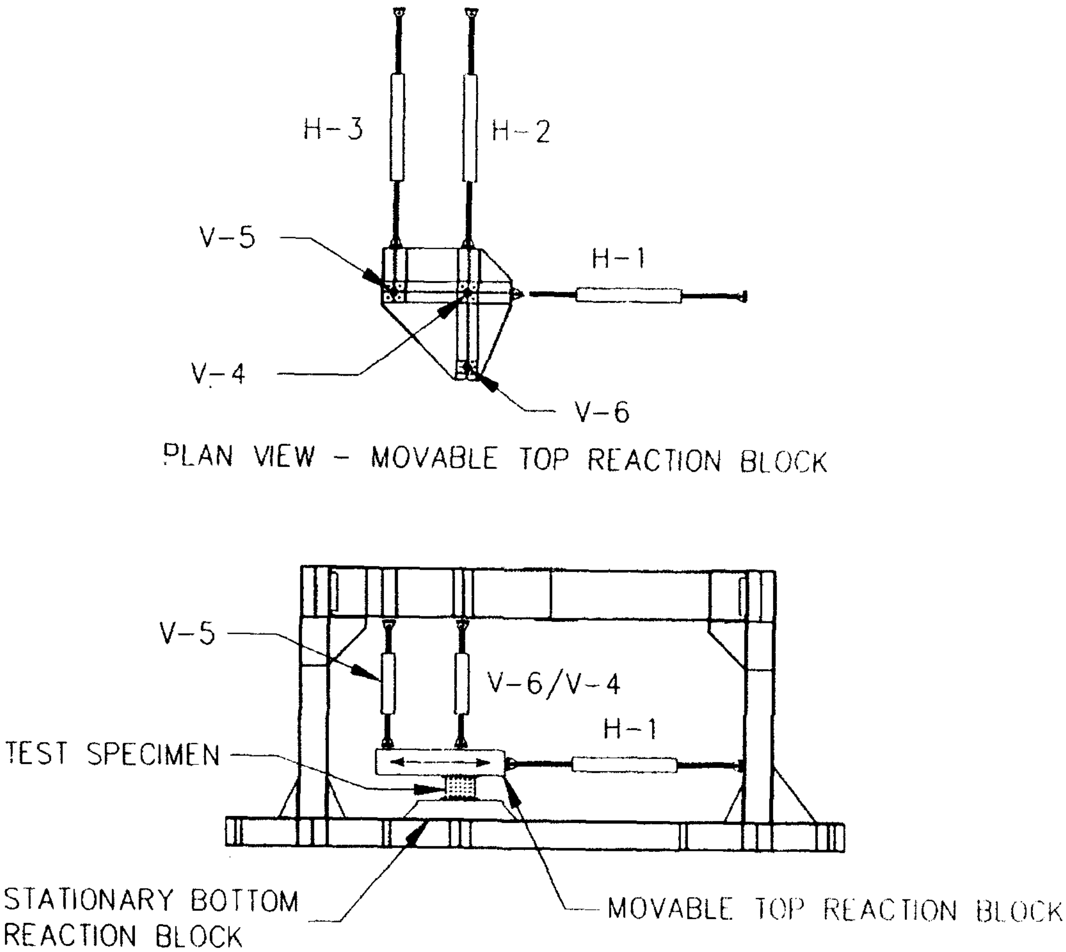

Figure 2. Test fixture actuator locations

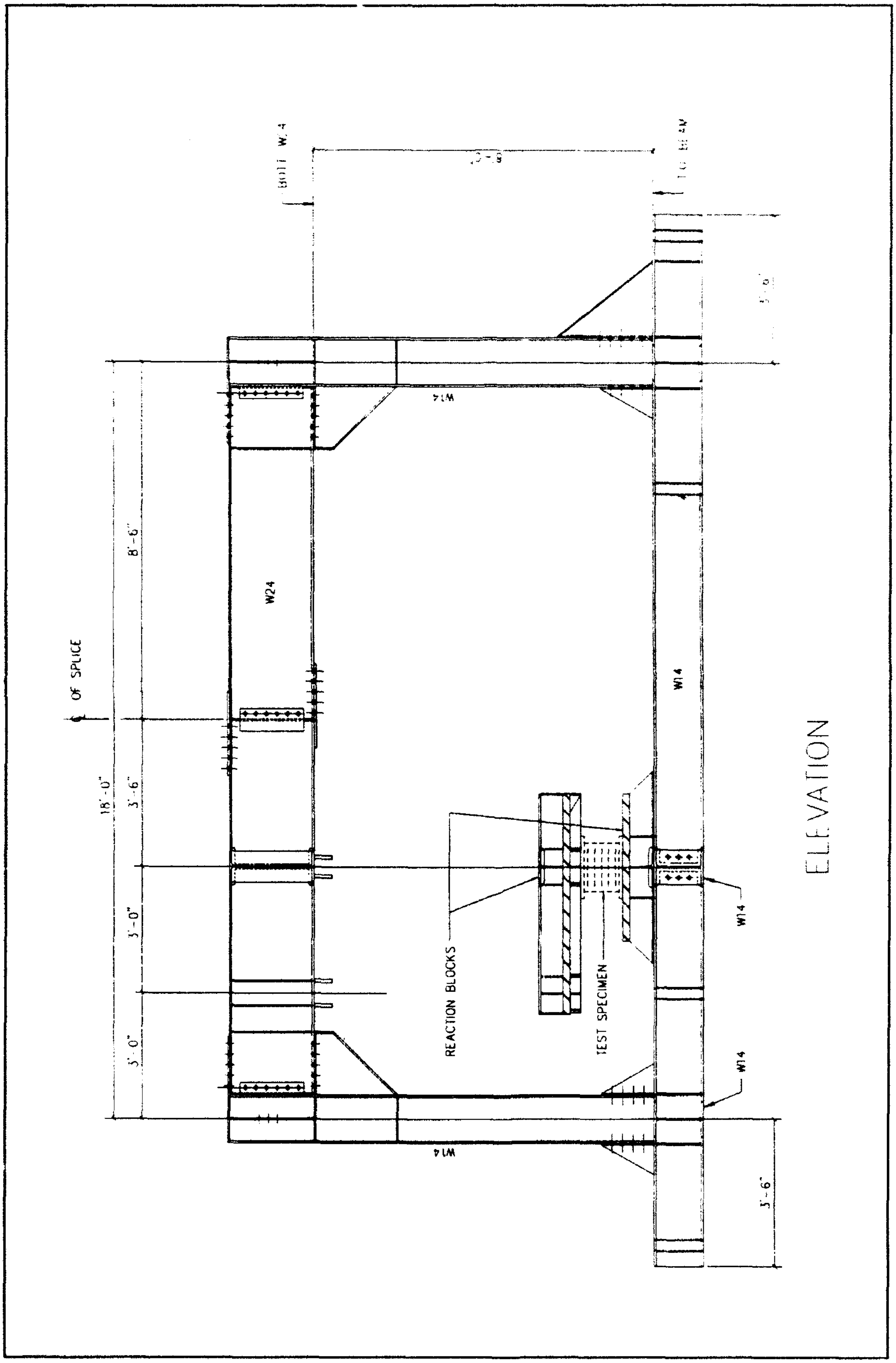

Figure 3. Test fixture elevation design concept

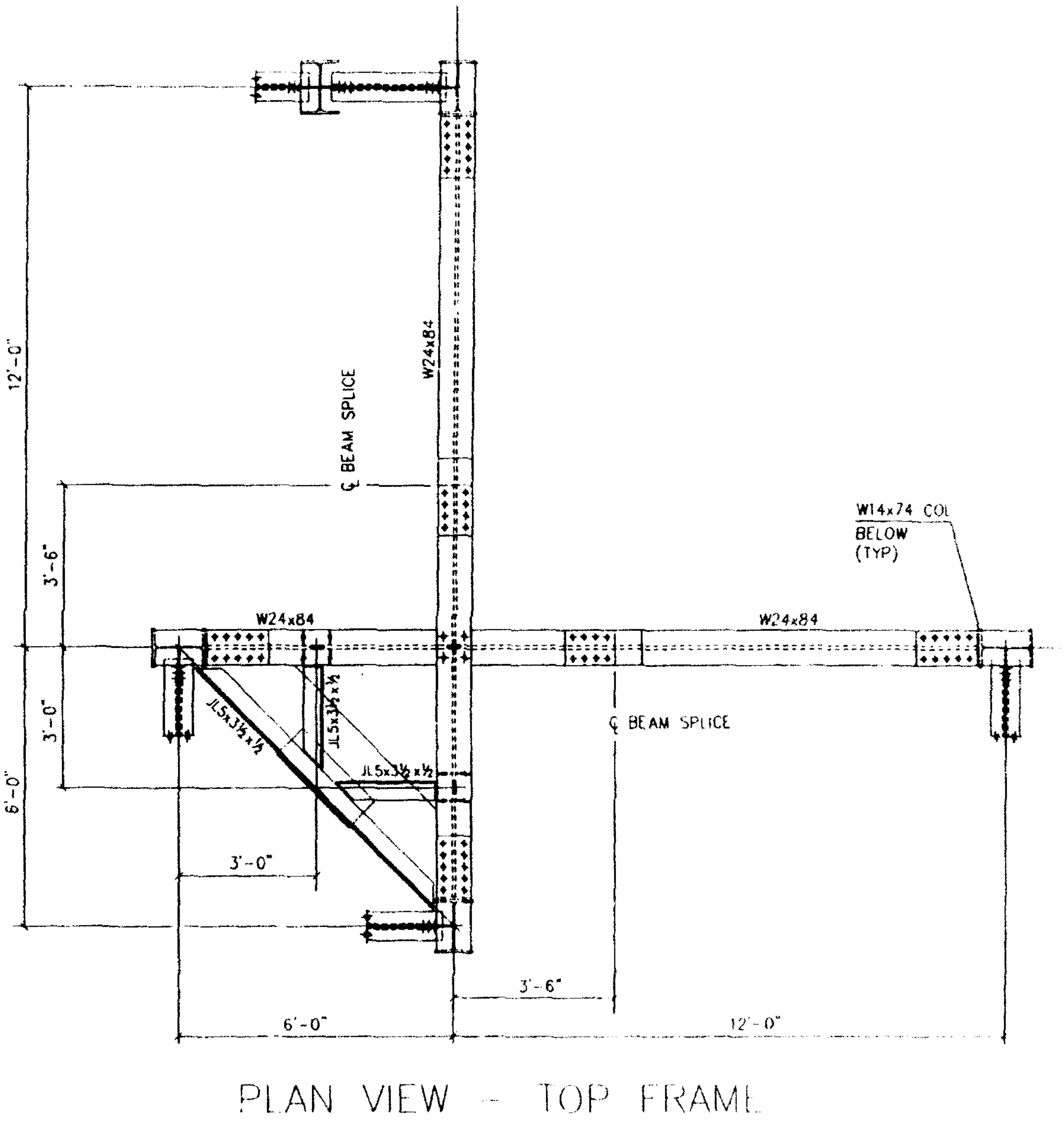

Figure 4. Test fixture top frame design concept

Table 2 **Actuator Force and Stroke Requirements**			
		Required Performance	
Actuator	Orientation	Force, kips	Stroke, in.
1	Horizontal	±90	±16
2	Horizontal	±90	±16
3	Horizontal	±35	±16
4	Vertical	+220	±3
5	Vertical	±50	±3
6	Vertical	±50	±3

two structural-steel frames perpendicular to each other, and will support all of the actuators and the reaction points for the test isolator. The frame members are laterally braced at their connections and at the actuator mounting points. The base framing and the columns are composed of 14-in.-deep steel "W" sections. The columns have moment-resisting connections at the base and top frame connections. Lateral stability is provided by steel double-angle sections and plates at frame joints, and the actuator mounting points. Stiffener plates are used to strengthen the column webs at the load application points. The top framing beams are composed of 24-in.-deep steel "W" sections. The location where the two top framing beams intersect to form the mounting point for Actuator V-4 uses an all-welded construction. Two reaction blocks are used to secure the test isolator in position and to apply the test deflections. Each block is constructed with 2-in. steel plate stiffened with steel plates and steel WT sections.

Since the base isolators will be subject to large lateral deflections, the stability of the reaction frame had to be investigated under a variety of loading conditions. The maximum frame displacements occur at the actuator mounting points. Under the worst-case loading, the maximum deflection of the frame was found to be at the mounting location for the vertical actuator, and is approximately 0.16 in. Since the base isolators will be tested under cyclic and seismic loading, a dynamic analysis was performed to ensure that the natural vibration period of the reaction frame did not resonate with the applied excitation frequencies for the dynamic loading functions. The calculated vibration period for the first mode of the frame was 0.036 sec (28 hz) which will be well above the planned excitation frequencies and should not produce interaction with the frame.

The axial force and the deflection of each of the six actuators will be measured to define the input excitation and the magnitude of any rotational moments in the top reaction block. During the cyclic and seismic tests, three mutually perpendicular acceleration measurements will be made on the top of the top reaction block to determine the effects of the inertial forces caused by the mass of the top reaction block and the various other masses attached to the top of the isolator.

The displacement response of the isolator will be determined by measuring the relative location of the top reaction to the fixed bottom reaction block. Four direct current displacement transducers (DCDTs) will be mounted vertically between the four corners of the top and bottom reaction blocks. Eight DCDTs will be mounted diagonally between the two reaction blocks to measure lateral displacement and rotation of the top of the isolator.

Test Program

The stated objective of the research project is to develop comparative data on various types of base isolators to provide a data base and design guidelines for the application of base isolation systems. To accomplish this goal, USACERL will solicit candidate base isolators from all of the major manufacturers for experimental static, cyclic, and seismic testing in both biaxial and triaxial environments. Comparative testing will be performed to investigate the effects of various isolator parameters on the isolation performance and to determine the interaction between these parameters.

The majority of available base isolation systems fall into two main types, elastomeric systems and various types of sliding systems. The dynamic performance of a base isolated structure can be influenced by a number of

parameters that are different for the two types of isolation systems. For elastomeric isolators, these parameters include axial stress and strain, shear stress and strain, equivalent viscous damping (horizontal and vertical), loading frequency, stability (buckling and roll-out), and low-cycle fatigue. For sliding isolation system systems, these parameters include contact pressure, sliding velocity, and static and dynamic coefficients of friction. All of these parameters will be investigated during the testing program in the triaxial test fixture. A number of parameters for elastomeric isolators will also be determined using a 1000-kip Tension/Compression load machine including axial stiffness, tensile strength, and vertical damping characteristics at large axial strain and zero percent shear strain.

Current Status

As part of Phase I of this research effort, all of the known base isolation manufacturers and several respected researchers in the area of base isolation research were contacted concerning their interest in contributing to the comparative base isolation testing program. Of the eleven individuals and companies initially contacted, currently eight have replied expressing a willingness to participate. Five manufacturers have agreed to supply candidate isolators for testing at no charge to the government.

The cost to construct the test frame and purchase the required hydraulic and electronic components is estimated at approximately \$500k. This cost is too high to be supported solely by DOD, but USACERL has approached several other government agencies with large inventories of buildings in areas of seismic risk about the possibilities of joint sponsorship of the work. These agencies include Veterans' Affairs, Department of Energy, and Government Services Administration.

One task related to the test fixture that remains to be accomplished is the development of control algorithms to operate the six hydraulic actuators This will be a significant effort because of the complexity of the interactions between the six actuators attempting to control six degrees of freedom of motion. This work will be accomplished through a cooperative effort with the University of Illinois.

The test results will be used to develop design and selection guidance for the application of base isolation technology in the DOD, other government agencies, and the public sector.

Masonry Program Development Criteria

by
Harold C. Thomas, Jr.[1]

Abstract

One of the problems facing design engineers and the masonry industry today is the lack of automated design tools available to engineers and contractors to simplify the design process of designing masonry structures. In an attempt to address this problem within the Corps of Engineers, the Computer-Aided Structural Engineering (CASE) Masonry Task Group is currently developing a series of masonry programs to do specific design tasks for the design engineer. Among the types of designs to be developed include bearing wall, lintel, pilaster, edge stiffener, shear wall, and shear wall rigidity determination.

One of the key features of the programs is that they will be menu driven and user friendly, yet very powerful. Once these programs are developed, they will be combined into one program containing all of the design options of each individual program. Eventually this program will be incorporated into the Computer-Aided Structural Modeling Package (CASM) now currently under development.

This presentation will give an overview of the committee's work in the development of these programs and an explanation of the individual features of each.

Introduction

In August 1989 the Masonry Task Group was organized to provide the Corps of Engineers support for designing and building high quality masonry structures. The objectives of the group are to do the necessary development, research, and investigation to provide design and construction guidance and standards to engineers within the Corps of Engineers. In addition, the group's goals are to disseminate information on criteria, research, standards, nondestructive evaluation methods, and code-writing committee activities as they become available. The group also acts as the Corps of Engineers' point of contact on all masonry activities.

When the group was formed in 1989, we quickly identified the need to develop computer design aids within the Corps of Engineers to help engineers perform masonry designs quickly and with minimum error. The group researched the masonry community to determine what masonry design computer programs currently existed and were available to be used by the Corps of Engineers for design purposes. The group evaluated a number of programs but determined that either the criteria that these programs were based on did not coincide with Corps of Engineers' criteria, or the programs were not versatile enough to meet the needs of the design engineer. The group concluded that to have these kinds of design tools available to

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engineers within the Corps of Engineers we must produce programs internally to meet our specific needs. A procedure was established to accomplish this task within the limited time the group members had to devote to the effort. The decision was made that the task group would determine and produce the criteria documents that would be used to develop the programs. The actual development of the programs would be contracted to an outside source by the US Army Engineer Waterways Experiment Station.

Within every masonry building system there are a number of masonry design elements that must be considered to produce an adequately designed and well integrated system. The task group has identified and prioritized a number of masonry design elements to be supported by these programs. The following is a list of the masonry programs that were determined are needed within the Corps of Engineers:

- Walls.
- Columns.
- Pilasters.
- Edge stiffeners.
- Shear walls (including determination of wall rigidities).
- Lintels and beams.
- Strength design for tall slender walls.
- Strength design for shear walls.
- Strength design for all elements of masonry design when a formal strength design code is approved.

The plan to develop these programs is to produce individual stand-alone programs for each of the design tasks listed and to do so in a modular format so that they can be combined into one program in the future when all are completed. The combined program will have a central menu to choose the design option desired. There are several advantages of producing programs in this manner. As soon as an individual design element program is produced and approved it can be released to design engineers to use without having to await development and approval of the other design element programs. By this method we can place masonry computer-aided design into the hands of design engineers sooner than waiting for the entire combined program to be completed. It is a much simpler process to manage the development of an individual module at a time than several modules at once. In addition, budget constraints prevent awarding the size contract needed to produce the entire package at one time. This work and its funding will be spread out over several fiscal years.

Some of the key features of these programs include a menu-driven user-friendly environment, several levels of output options including graphics, and an on-line help system explaining criteria requirements. The menu-driven system will lead the engineer step by step through the decision-making process providing the user with different options along the way. An on-line help system will assist the user in deciding what options to use in the design by giving explanations of the criteria requirements for each step in the design process. The user will also have the option of generating output in either an abbreviated format giving only a summary of the solution or an expanded format which generates output similar to that performed by hand calculations showing individual equations and answers step by step in arriving at the solution.

These masonry programs will be developed in a sequence based on those design elements used most frequently and in the greatest demand by users. The first programs to be developed are "Design of Concrete Masonry Bearing Walls" and "Design of Hollow Unit Masonry Lintels and Beams," respectively. The criteria documents for those two programs have been included as a part of this paper. A schedule for completion of the first two programs has not been established, but it is the priority of the Masonry Task Group to place these programs in the hands of design engineers as soon as they can be developed.

Design of Concrete Masonry Bearing Walls

Purpose

The purpose of this document is to establish the criteria for development of a computer program for the design and review of reinforced single wythe hollow unit masonry bearing walls. The program will design masonry bearing walls by the working stress method for both axial loads and moments or any combination thereof applied at the ends of the wall as well as wind, seismic, or other loads perpendicular to the plane of the wall. The program will not be capable of designing for in-plane shear loads. Design is based on one-way spanning of walls.

Capabilities

The program will be capable of design or review of any reinforced or unreinforced concrete masonry bearing walls using standard-size concrete masonry units, from 4 to 16 in. thick. The wall will be assumed to be supported to resist out-of-plane loading at the top and bottom of the wall. The wall is analyzed using working stress methods and will consider load-deflection (P-Delta) effects. Analysis is done in accordance with TM 5-809-3 (draft revision in preparation) and TM 5-809-10 (Department of the Army 1982). This is similar to the method used for analysis of a T-beam. The program will be designed as a menu-driven interactive program, easy to use for the beginner yet efficient for the experienced user. To the maximum extent possible, the input and output will be displayed graphically. The option should be available to print out the results long hand similar to what we would see had the calculations been done by hand.

Program architecture

This program is the first of many programs for the design/review of masonry components. Each program, when developed, will function as a "stand-alone" program and will ultimately serve as a module in an umbrella program encompassing design/review of masonry components. These programs will be written in a language that will benefit as much as possible inclusion of the masonry design option into the Computer-Aided Structural Modeling (CASM) program currently under development. The programs should be written to handle input/output (I/O) in a fashion that facilitates "stand-alone" or "integrated" usage. To facilitate this and make modifications due to masonry code changes easier, structured modular programming practices will be used with all I/O and code checking provisions will be separated from the program control structure.

Loads

Loads may consist of axial loads and external moments applied to the ends of the wall and forces applied perpendicular to the wall. Axial loads, whether dead or live, can be inputted either as concentrated loads, distributed loads, or a combination of the two. The program will consider the following loading combinations:

- Dead load.
- Dead load plus live load.
- Dead load plus wind or seismic.
- Dead load plus live load plus wind or seismic. (User should be able to input the percentage of live load desired for this load case).
- Other.

Load combinations will be applied with the appropriate factors in accordance with TM 5-809-1 (Department of the Army 1986) or may be combined with user-specified factors. Self weight (dead load) of the wall is computed based on the geometry and weight given in TM 5-809-3 (draft revision in preparation) and the stiffener (or grouted cell) spacing specified by the user or determined by the program. Out-of-plane seismic loads resulting from the dead weight of the wall will be computed by the program based on the inputted

seismic zone and in accordance with TM 5-809-10 (Department of the Army 1982). The user should have the option of choosing which loading case or cases he prefers to design for.

Method of analysis and design

This program will design or review the design of walls using working stress methods and the stress ratio interaction equation (unity equation) for combined axial and bending stresses. The user will have available the option to use the P-M (axial compression - moment) interaction equation in lieu of the stress ratio interaction equation. The P-M interaction equation is less conservative but recognizes the combined effect of compression (P) and moment (M). In the design mode, the wall will be proportioned to satisfy the following requirement for all loading cases:

$$\frac{fa}{Fa} + \frac{fb}{fB} < 1.0 \text{ (or 1.33 for wind or seismic combinations)}$$

In the review mode the program will calculate and display the interaction equation for each load case. In the design mode analysis for design will begin with initial minimum thickness and reinforcement requirements as required by TM 5-809-10 (Department of the Army 1982), Tables 8-4 and 8-5. Reinforcing steel bar sizes will be determined based on the minimum area of steel required, the maximum allowable bar spacing and 8-in. modular spacing. Reinforcing sizes will be "rounded" to the next larger bar size meeting the minimum area of steel requirements for seismic. Stresses will be computed for the required load combinations. If the stresses are not within the requirements of the interaction equation, then the reinforcing steel area will be incremented to the next larger bar size. Stresses will be recomputed and compared again with the interaction equation. The reinforcing steel area will be increased one bar size per iteration until the interaction equation is satisfied or the maximum allowable bar size is reached. If the maximum allowable bar size does not satisfy the interaction equation, then the stiffener spacing will be reduced by 8 in. The bar size will be set at number 4 or the bar size required to meet the minimum area of steel required, whichever is larger. Stresses will be computed and compared with the allowable stresses and the interaction equation, as previously stated. The iteration process will then repeat as stated for each successive bar size and stiffener spacing until the interaction equation is satisfied. If the minimum (initial) wall thickness cannot satisfy the interaction equation, with all cells reinforced (the user can also input the minimum steel spacing preferred), then the wall thickness will be incremented to the next size concrete masonry unit. The process will repeat until the interaction equation is satisfied.

- **Design equations**. The equations for the design of reinforced and unreinforced walls are given in the appendix and were obtained from TM 5-809-3 (draft revision in preparation) and TM 5-809-10 (Department of the Army 1982).
- **Allowable stresses**. All allowable stresses are shown in the appendix. These allowable stresses were obtained from Chapter 4 of TM 5-809-3 and Tables 8-2 and 8-3 of TM 5-809-10.
- **Actual stresses**. Actual stresses will be computed using the equations in the appendix as well as those in Chapter 4 of TM 5-809-3 (draft revision in preparation).

Program input

Input to this program will be designed to be as user friendly as possible while maintaining the capabilities and flexibility needed by experience designers. The program will allow the user to build an input file interactively without extensive use of external references. Help screens should be available for each input item to assist the user in a clear understanding of the requested input. The help screens should include graphic representations of the input parameter wherever appropriate. Input screens will include the common terminology, the common symbol, and the required units for the input variable. Examples of possible input screens are

screens 1, 2, 3D, and 3R, attached. Program input will be saved as a file to recall and modify at a later date.

Program output

An example of a possible output screen is screen 4. Input and output files will be created from the screen input and output. The input files should be recallable to use as seed files to create a new file. Input will be echoed on the screen and written to output files. Output text and graphics files should be recallable for "replay." In review mode the portion of criteria not met will be identified.

Design of Hollow Unit Masonry Lintels and Beams

Purpose

The purpose of this document is to establish the criteria for development of a computer program for the design and review of reinforced single wythe hollow unit masonry beams. This program will be designed to work similar to the Masonry Bearing Wall Program. The program will design masonry beams by the working stress method for inplane loads applied along the length of the beam and moments applied at the ends of the beam. The program will also be capable of reviewing for out-of-plane loads (wind and seismic loads).

Capabilities

The program will be capable of design or review of any reinforced concrete masonry beam using standard-size concrete masonry units, from 4 to 16 in. thick. End conditions for the masonry beam can be specified as simple or fixed. The beam must be laterally supported at intervals not to exceed 32 times the least width of the compression face. Deflections will be computed using the effective moment of inertia. The program will be designed as a menu-driven interactive program, easy to use for the beginner yet efficient for the experienced user. To the maximum extent possible, the input and output will be displayed graphically. The option should be available to print out the results long hand similar to what we would see had the calculations been done by hand.

Program architecture

This program is one of many programs for the design/review of masonry components. Each program, when developed, will function as a "stand-alone" program and will ultimately serve as a module in an umbrella program encompassing design/review of masonry components. These programs will be written in a language that will benefit as much as possible inclusion of the masonry design option into the CASM program currently under development. The programs should be written in a compatible computer language and handle input/output in a fashion that facilitates "stand-alone" or "integrated" usage. To facilitate this and make modifications due to masonry code changes easier, structured modular programming practices will be used with all I/O and code checking provisions separated from the program control structure.

Loads

Vertical and lateral loads may consist of specified loads or computed lintel loads. Loading on the masonry beam may be specified as:

- Uniform loads.
- Triangular loads.
- Up to 10 concentrated loads.
- Load combinations of all or part of the three specific loads.

Lintel loads may be computed by the program based on the height of the wall above the lintel. Distributed and/or concentrated loads may be specified at the top of the wall under consideration. Arching action will be assumed by the program if the height of the wall above the lintel is equal to or greater than 1/2 the clear span of the lintel. Arching action is assumed to spread loads through a

SCREEN 1 (Input)

BEARING WALL INPUT

Project:
Location:
Engineer:

Input File Name: (If name is given file will be saved)
Wall identifier:

Wall Height, h :_____ft. (Clear Height Between Support Points)
Axial Load, P :_____ lb./ft.
Eccentric Axial Load, Pe :_____ lb./ft.
Eccentric Axial Load Eccentricity, e :_____ in. from centerline.
Wind Load, w :______ psf
Seismic Zone, Z :____ (0, 1, 2, 3, or 4).
Importance Factor, I :_____

Other Loads:
Concentrated Axial:
P = _____ lbs.
Spacing = ______ in.
Line Load, Axial:
plf = ______
Distributed Moment at Top of Wall:
Mt = _______ ft-lb/ft
Distributed Moment at Bottom of Wall:
Mb = ________ ft-lb/ft
Concentrated Lateral Load:
l = _____ lbs.
Loction = ________ ft. from bottom of wall

SCREEN 2 (Computer response and input)

Wall height, h = echo ft.
Seismic Zone, Z = echo
Wind Load, w = echo psf

Minimun thickness, t = (TM value)
Max. spacing of vertical reinforcing, s = (TM value)
Max. spacing of horizontal reinforcing, sh =(TM value)

Max. Moment, Mmax =(computed value) ft.-lb.

DO YOU WANT TO DESIGN OR REVIEW ? (D or R)

SCREEN 3D (Input, Response = DESIGN)

Wall height, h = echo ft.
Seismic Zone, Z = echo
Wind Load, w = echo psf

Minimum thickness, t = (TM value)
Max. spacing of vertical reinforcing, s = (TM value)
Max. spacing of horizontal reinforcing, sh =(TM value)
Max. reinforcing bar size, No. = (Based on t)
Minimum spacing of vertical reinforcing, sm = ______.

Max. Moment, Mmax =(computed value) ft.-lb.
Max. Axial Load, Pmax =(computed value) lb./ft.

Design masonry f'm =(user input) psi.
Steel yield stress, fy =(user input) ksi.

REINFORCING STEEL WILL BE SPACED AT
8" MODULAR SPACING.

Grout all cells ? (Y or N)

Response = Yes: Go to next screen.
Response = No:
ONLY CELLS WITH REINFORCING STEEL
WILL BE ASSUMED TO BE GROUTED.

SCREEN 3R (Input, Response = REVIEW)

Wall height, h = echo ft.
Seismic Zone, Z = echo
Wind Load, w = echo psf

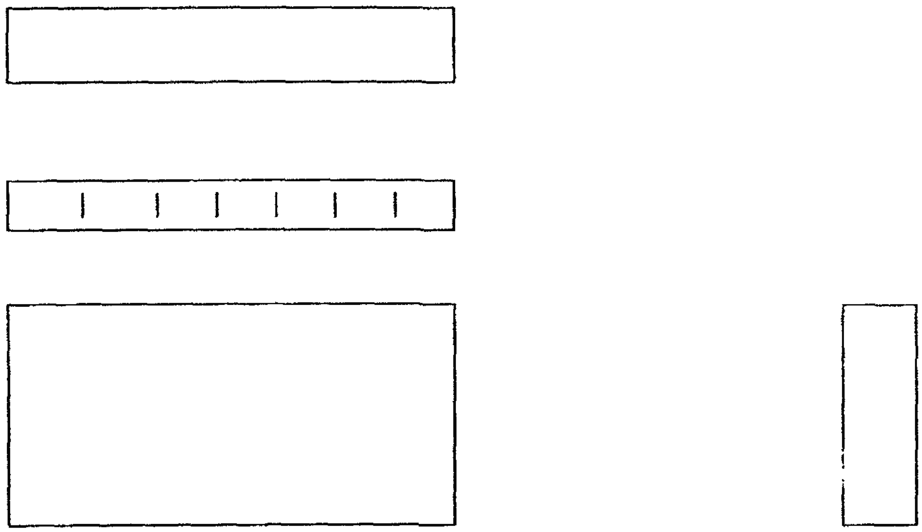
Minimum thickness, t = (TM value)
One Bar or Two Bars? _____. (Enter "1" or "2")
Max. spacing of vertical reinforcing, s = (TM value)
Max. spacing of horizontal reinforcing, sh = (TM value)
Max. reinforcing bar size, No. = (Based on TM 5-809-3, Draft)

Max. Moment, Mmax = (computed value) ft.-lb.
Max. Axial Load, Pmax = (computed value) lb./ft.

Design masonry f'm = (user input) psi.
Steel yield stress, fy = (user input) ksi.

Spacing of vertical reinforcing steel, s = (user input)
Area of steel/spacing, As = (user input)

SCREEN 4 (Output)



VERTICAL REINFORCING : #6 @ 16 in. ctrs. (computed)
HORIZONTIAL REINFORCING: #4 @ 48 in. ctrs. (computed)

Asv =(computed value) in2/ft, p = (computed value)
Ash =(computed value) in2/ft, p = (computed value)

Mmax =(computed value) ft-lb
Mrm =(computed value) ft-lb
Mrs =(computed value) ft-lb

$$\frac{fa}{Fa} + \frac{fb}{Fb} <= 1.0$$

$$\frac{\text{(computed)}}{\text{(computed)}} + \frac{\text{(computed)}}{\text{(computed)}} = \text{(computed value)}$$

triangular shaped distribution having sides sloping at 45 deg with the horizontal. Concentrated or distributed loads applied to the wall above the apex of the distribution triangle will not be added to the lintel load. The program will compute the dead load of the wall above the lintel (including arching action where appropriate) and the dead weight of the lintel itself based on the specified depth of the lintel. Load combinations will be applied with the appropriate combinations and factors in accordance with TM-5-809-1 or may be combined with user-specified factors. Self-weight (dead load) of the wall is computed based on the geometry and weight given in the draft revision of TM-5-809-3 (in preparation). Out-of-plane seismic loads resulting from the dead weight of the wall will be computed by the program based on the input seismic zone and TM-5-809-10 (Department of Army 1982).

Method of analysis and design

This program will design or review the design of masonry beams using working stress methods. The procedure used will follow the equations and design process as described in TM-5-809-3, Chapter 8, "Lintels." In either the design or review mode, the program will calculate and display the compressive stress in the masonry and the tensile stress in reinforcing steel as well as the stress ratio for each. In the design mode, analysis for design will begin with the initial minimum thickness and reinforcement requirements as specified by the engineer. Reinforcing steel bar sizes will be determined based on the minimum area of steel required. Reinforcing sizes will be "rounded" to the next larger bar size meeting the minimum area of steel requirements. Stresses will be computed for the required load combinations. If the stresses do not satisfy the allowable stresses, then the reinforcing steel area will be incremented to the next larger bar size. Stresses will be recomputed and compared again with the allowable stresses. The reinforcing steel area will be increased one bar size per iteration until either the actual stresses are less than or equal to the allowable stresses or the maximum allowable bar size is reached. If the maximum allowable reinforcing steel does not satisfy the allowable stress requirements, then the depth of the lintel or beam will be increased by 8 in. The bar size will be set at No. 4. Stresses will be recomputed and compared with the allowable stresses, as stated. If the allowables are not met, the iteration process will then repeat as above for each successive bar size until the allowable stress requirements are satisfied. The process will repeat until the stress equations are satisfied.

- **Design Equations**. The equations for the design of reinforced walls are given in the appendix and were obtained from TM 5-809-3 (draft revision in preparation) and Chapter 5, TM 5-809-10 (Department of the Army 1982).
- **Allowable Stresses**. All allowable stresses are shown in the appendix. These allowable stresses were obtained from Chapter 5, draft revision, TM 5-809-3.
- **Actual Stresses**. Actual stresses will be computed using the equations in the appendix as well as those in Chapter 8, draft revision, TM 5-809-3.

Program input

Input to this program will be designed to be as user friendly as possible while maintaining the capabilities and flexibility needed by experience designers. The program will allow the user to build an input file interactively without extensive use of external references. Help screens should be available for each input item to assist the user in a clear understanding of the requested input. The help screens should include graphic representations of the input parameter wherever appropriate. Input screens will include the common terminology, the common symbol, and the required units for the input variable. Input should appear similar to the attached example input screens. Program input will be saved as a file in order to recall and modify at a later date.

Program output

Output should look similar to the attached example output sceens. Input and output files will be created from the screen input and output. The input files should be recallable to use as seed files to create a new file. Input will be echoed on the screen and written to output files. Output text and graphics files should be recallable for "replay." In review mode the portion of criteria not met will be identified.

References

Headquarters, Department of the Army. 1982. "Seismic Design for Buildings," Washington, DC.

Headquarters, Department of the Army. 1986. "Load Assumptions for Buildings," TM 5-809-1, Washington, DC.

Headquarters, Department of the Army. "Masonry Structural Design for Buildings," TM 5-809-3 (draft revision in preparation), Washington, DC.

Fracture Analysis of Lock Wall

by

Prof. Victor Saouma[1]

(Copy of paper not available)

[1] University of Colorado.

Black Rock Lock Stability and Foundation Problems and Solutions

by

Eugene N. Lenhardt[1] *and Frank T. Lewandowski, PE*[2]

Abstract

The Black Rock Lock located on the Niagara River in Buffalo, New York, was recently found to have both stability and foundation problems. The lock chamber walls do not meet required overturning stability criteria when the lock chamber is dewatered. Numerous voids have been discovered within the bedrock foundation under the lock walls. The Buffalo District has conducted thorough studies of these problems and looked at various solutions. These studies and their recommendations for rehabilitation of the Black Rock Lock are discussed in this paper.

Introduction

The River and Harbor Act of 3 March 1905 provided for a suitable deep-draft channel around the rapids and shoals at the head of the Niagara River at Buffalo, New York. The act included the construction of a navigation lock, bridge, and repair of existing piers and walls. The Black Rock Lock, constructed by the Buffalo District Corps of Engineers between 1908 and 1913, was opened to deep-draft vessel traffic on 17 August 1914. The Black Rock Lock is located on the right bank of the Niagara River approximately 4 miles downstream from the head of the river at Lake Erie. Location, plan, profile, and sections of the Black Rock Lock are shown in Figure 1. Currently, both lock wall stability and foundation problems exist at the Black Rock Lock. Stability problems involve the lock chamber walls between miter gate monoliths (Section B-B in Figure 1). These lock walls do not meet current Corps of Engineers overturning stability criteria when the lock chamber is dewatered. Foundation problems consist of a high degree of solutioning and weathering of the lock wall bedrock foundation making it difficult to dewater the lock chamber and subjecting the lock walls to undermining. This paper will discuss the studies conducted by the District relative to the lock wall stability and foundation problems and all the various solutions to these problems that were considered including the selected rehabilitation plan.

Background

Throughout its life the lock chamber of the Black Rock Lock has been successfully dewatered without incident. However, by the early 1980's, the lock chamber was becoming increasingly more difficult to pump out. Seepage, somewhere below the lock walls and miter gate sills, was suspected. In 1984, District engineers performed preliminary stability analyses in case it was necessary to dewater below the historical pump-out elevation (top of lower miter gate sill) in order to remedy the seepage problem. The preliminary stability

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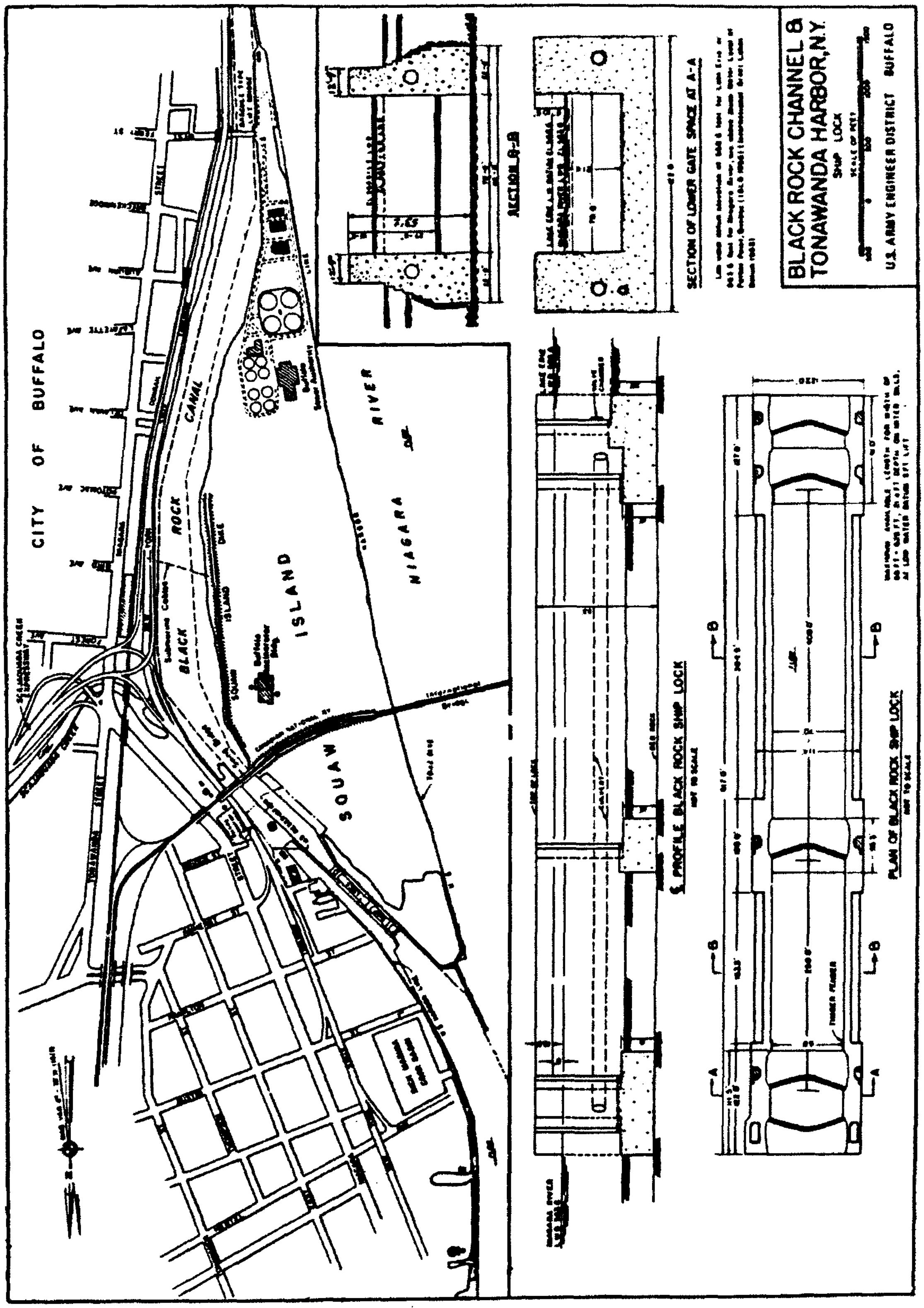

Figure 1. Location, plan, profile, and sections of the Black Rock Lock

analysis showed that the lock chamber walls between gates would not meet current Corps of Engineers overturning stability criteria for the dewatered condition when the chamber pool was at and below the historical pump-out elevation.

From 1986 to 1990, Hanson Engineers, Inc. of Springfield, Illinois, was under contract to the District to perform complete overturning and sliding stability analyses for all applicable loading conditions and to conduct a foundation investigation at the Black Rock Lock. Hanson's stability analyses verified that the lock walls did not meet current Corps of Engineers overturning stability criteria for the dewatered condition. It was, therefore, decided that, during all future dewaterings, the minimum pump-out elevation within the lock chamber would be kept at the historical level (top of lower miter gate sill). Hanson's foundation investigation found numerous voids under the lock walls. The voids were created by the solutioning of gypsum deposits and weathering of the bedrock foundation over the years. This was determined to be the cause of the lock chamber dewatering problems. Hanson was then requested to perform additional subsurface investigations including a test grouting program to use as the basis for a production grouting contract. In 1990, Hanson prepared contract plans and specifications for grouting the bedrock foundation of the Black Rock Lock. A contract for grouting the bedrock foundation of the Black Rock Lock was awarded during May 1991. Future dewatering of the lock chamber is being postponed until the grouting contract is completed. In 1990, using the results of Hanson's stability studies, Buffalo District engineers looked at various alternatives to solve the lock wall stability problems whenever the lock chamber is dewatered. The foundation investigations, test grouting program, the current foundation grouting contract, and the alternatives considered to remedy the lock wall stability problems are discussed in the remaining sections of this paper.

Subsurface Explorations

Winter 1988 exploration program

In January and February 1988, a subsurface exploration program was performed to determine the condition of the rock foundation and to obtain samples for laboratory testing. The locations of these explorations are shown in Figure 2.

The borings reveal that bedrock is at a uniform depth of 51 to 55 feet below the top of the lock wall. The bedrock consists of relatively flat-lying layers of dolomitic limestone and gypsum. The upper 5 to 10 feet of the bedrock was highly fractured and weathered with gypsum being rarely recovered in this zone. At depths below 8 to 15 feet, the bedrock was usually recovered intact and with some pure gypsum beds as much as 0.5 feet thick. The upper 5 to 10 feet of the highly weathered bedrock contained softened layers of gypsum intercepted by open vertical joints making the gypsum prone to rapid dissolution by flowing water. The dissolution of gypsum along the open vertical joints has most likely resulted in an extensive network of isolated and intersecting small cavities beneath the lock wall. A borehole video camera confirmed the existence of cavities beneath the lock wall some as much as 1-1/2 foot thick.

Summer 1988 test grouting program

Significant core loss and voids encountered during the winter 1988 subsurface exploration program indicated a higher degree of solutioning or weathering of the bedrock than previously expected. Concern that solutioning could undermine the lock prompted a second exploration program combined with a test grouting program that was performed during the months of June through September 1988. The objectives of the test grouting program were to:

- Estimate the extent of void formation and ability to fill large voids and rock fractures with grout.
- Evaluate different grout mixes (water-cement ratio), fillers (sand, rock flour), fillers/lubricants (fly ash, bentonite/clay), and accelerator additives (calcium chloride).
- Experiment with different grout pressures to establish suitable pressures for grout transmicity and filling of defects in the foundation.
- Develop reasonable criteria for spacing and orientation of the grout holes.
- Estimate the quantity of cement and other materials, drilling footage, and production rates to make cost estimates for production grouting.

The test grout program consisted of 3 test sections, designed to assess grouting conditions for a range of ground conditions. The locations of these test sections are shown in Figure 2. Test section A was located on the west wall near boring C88-8 in which the rock was highly weathered and fractured. In addition, old construction photos and subsequent maintenance dewatering showed this to be an area of high water inflows. The other test sections were located in areas thought to have better foundation conditions.

The number of grout injection holes in each test section were initially limited to six holes and were expanded to seven holes in sections B and C to provide sufficient information and to accomplish the project objectives. The locations of the grout injection holes for each test section are shown in Figures 3 and 4. All grout injection holes used an NX-sized (4-inch ID) double tube core barrel with diamond impregnated drill bits.

Because of the anticipated presence of voids or highly fractured rock, downstage drilling and grouting was performed in 3-5-foot stages. This method was used as it was anticipated that grouting the upper stage would stabilize the highly fractured rock immediately below the base of the lock walls, thus preventing the hole from caving.

Drilling and grouting was performed in numerical order of borings and stages. Grout injection holes 1, 2, and 3 (Figures 3 and 4) formed a grout curtain along the inside of the wall and served to confine grout injected in successive holes beneath the lock wall. Grout holes 4 in test section A (Figure 3), 4 and 5 in test sections B and C (Figure 4) acted as primary blanket grout holes that served to further confine grout beneath the lock walls. Grout hole 5 in test section A and 6 in test sections B and C acted as secondary holes, filling the gaps between the lock curtain holes and the blanket primary holes. Grout holes 6 in test section A and 7 in test sections B and C were used as tertiary and check holes. The primary curtain and blanket holes were spaced at 12 feet from each other. The middle curtain holes and secondary blanket holes were split spaced between the primary holes at a spacing of 6 feet. The tertiary and check holes were further split generally at spacings of 2.5 to 3 feet from adjacent primary and secondary grout holes.

Water and type II portland cement were the major components of the grout mixes. Other grout constituents included sand, fly ash, bentonite, and calcium chloride. A water to cement ratio (by weight) of 1:1 was generally pumped first followed by 0.75:1 and 0.5:1 as pumping and grout takes permitted.

Seams of grout were only recovered in adjacent holes and in some cases only partial recovery of grout was obtained from a previously grouted stage. Flushing and dilution of grout by lock cycling during the test grout program may be a source of the low grout recovery. Field observations indicate that the addition of fly ash to the grout mix resulted in longer set times. In several grout injection holes recovered grout cores containing fly ash had only set to a very low strength 15 hours after injection. Thus, based upon this experience and the longer set times, it was decided that fly ash should not be used in the production grouting program.

The average grout take versus hole sequence is shown in Figure 5. Generally, stage 1 had the highest grout takes with an average of 43.4 sacks of cement and ranges from 1 to 144 sacks of cement. This is interpreted to reflect the higher degree of fracturing and void formation in the upper 5 feet of the rock mass. The third stage had the lowest grout takes ranging from 0 to 3 sacks of cement. This reflects the tight, unfractured, and unsolutioned nature of the rock located 10 to 15 feet below the base of the lock. Stage 2 had intermediate takes ranging from 2 to 68 sacks of cement with an average of 18.8 sacks. Average grout takes reduced in going from primary to secondary to tertiary grout holes. In general, the grout takes in the secondary holes were 30 to 80 percent of the primary holes. Tertiary grout holes had grout takes of 0 to 30 percent of the primary holes.

Foundation Production Grouting

The production grouting program for restabilization of the lock walls and gate sills is designed to improve the bearing capacity of the rock foundation and improve the sliding resistance by filling large voids and cavities that have resulted from continuous solutioning of gypsum since lock construction. In addition, grouting the voids would reduce seepage into the lock which has made dewatering impossible in recent years. Data from the summer of 1988 test grouting program provided useful information that was used to plan this program.

Grout materials and proportioning

The recommended or specified grout mix consists of the following materials:

- Type II Portland Cement, manufactured to resist moderate sulfate attack.
- Clear, clean, fresh mix water.
- Fine masonry sand to be used as a low cost filler and to promote plugging of large voids.
- Bentonite may be added (maximum 2 percent by weight) to reduce shrinkage, improve pumpability, prevent bleeding, and limit premature settling of solids.
- Accelerator (calcium chloride) added to reduce grout set time. A maximum of 2 percent by weight of accelerator may be added to the grout mix.

Fly ash will not be permitted to be used in the grout mix as a filler material. This restriction is based upon the low grout strengths and longer set times obtained from grout containing fly ash during the test grout program.

The grout shall be batched at incremental water to cement ratios of 2:1, 1:1, 0.75:1, and 0.5:1 by weight. Initially a grout mix ratio of 1:1 shall be injected. Depending upon the thickness and extent of void formation, the grout mix may be thickened to grout mix ratios of 0.75:1 and 0.5:1. If tighter rock conditions are encountered, the grout mix shall be thinned to a 2:1 water to cement ratio—then, depending upon grout pressures developed and grout takes, thickened to subsequent water to cement ratios within the sequence of 1:1, 0.75:1, and 0.5:1. Based upon the results of the test grouting program, it is expected that an initial grout mix of 1:1 will be injected with the majority of the grout being injected at a mix ratio of 0.75:1.

Grout injection procedures

Due to the existence of voids and highly fractured rock in the upper portions of the rock, downstage drilling and grouting will be performed. The grouting will be conducted in two successively deeper 5-foot stages as the cavity formation and rock fracturing primarily occurs in the upper 5 to 10 feet of the bedrock. After each stage is drilled, the grout hole will be washed and water pressure tested. Grout will then be injected and allowed to set for 24 hours before the next stage is drilled and the sequence repeated.

Grout pressures

The test grouting program revealed that safe grout pressures well below the maximum

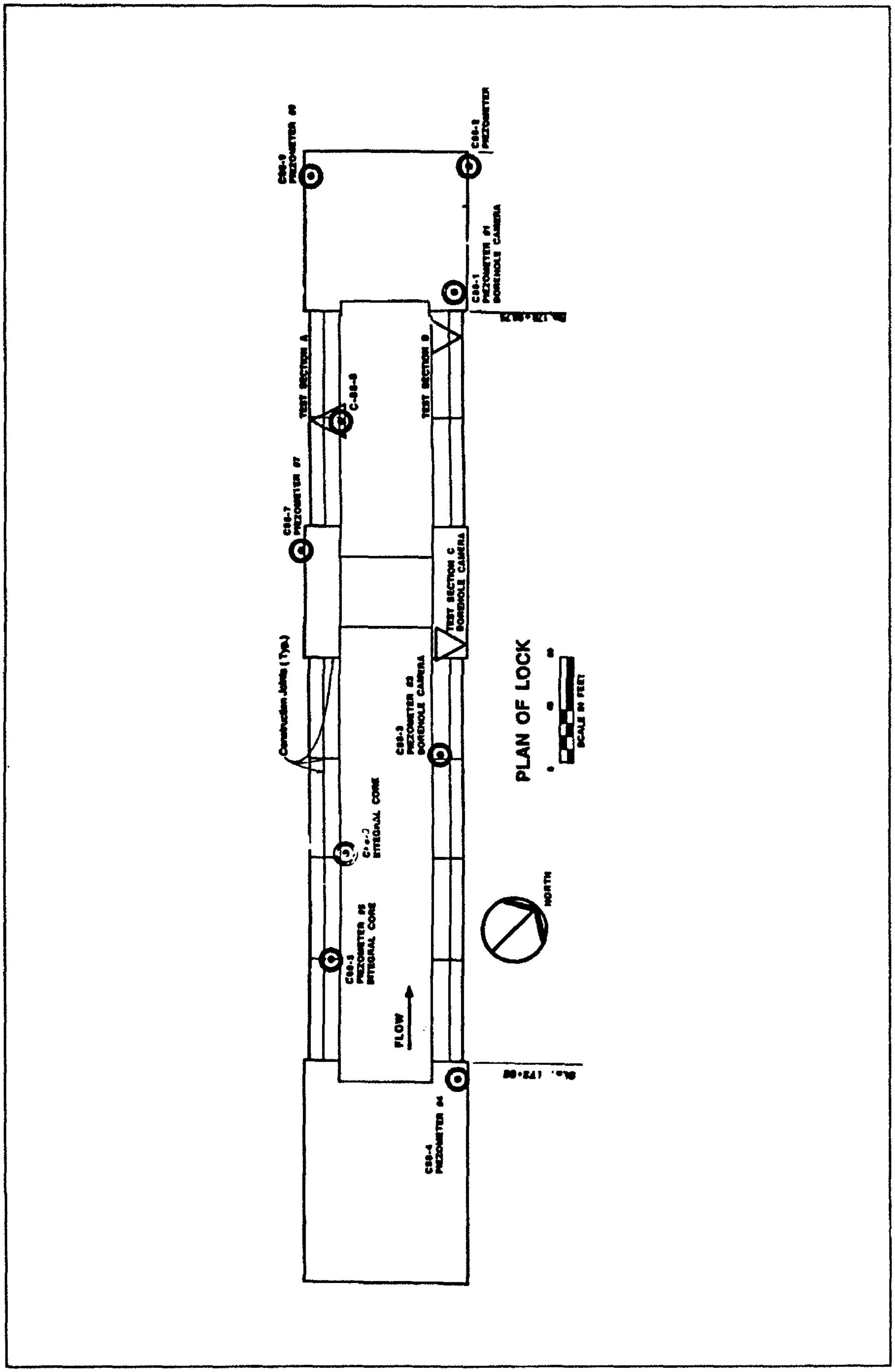

Figure 2. 1988 Plan of subsurface explorations and location of test grout sections

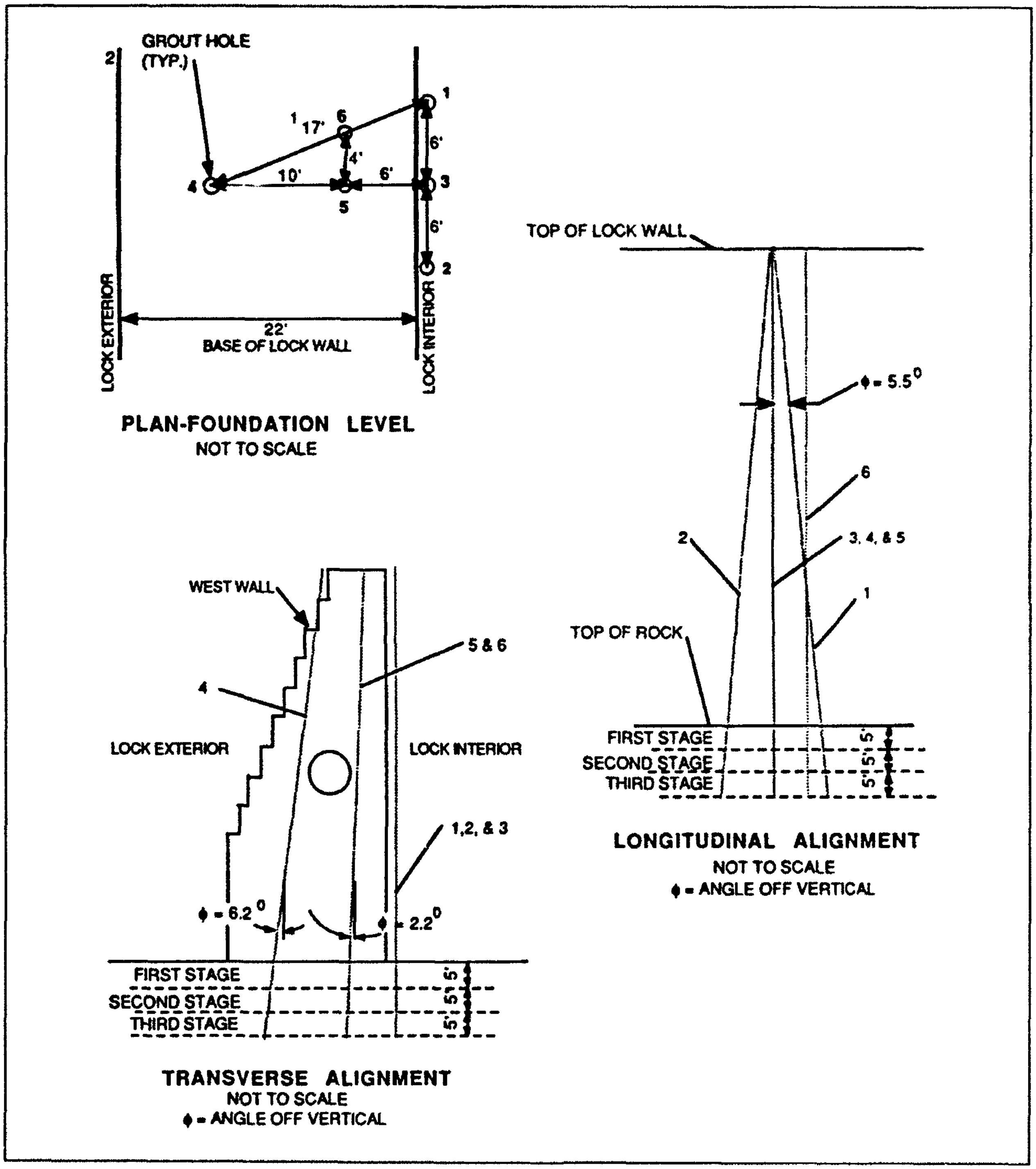

Figure 3. Test grout section A

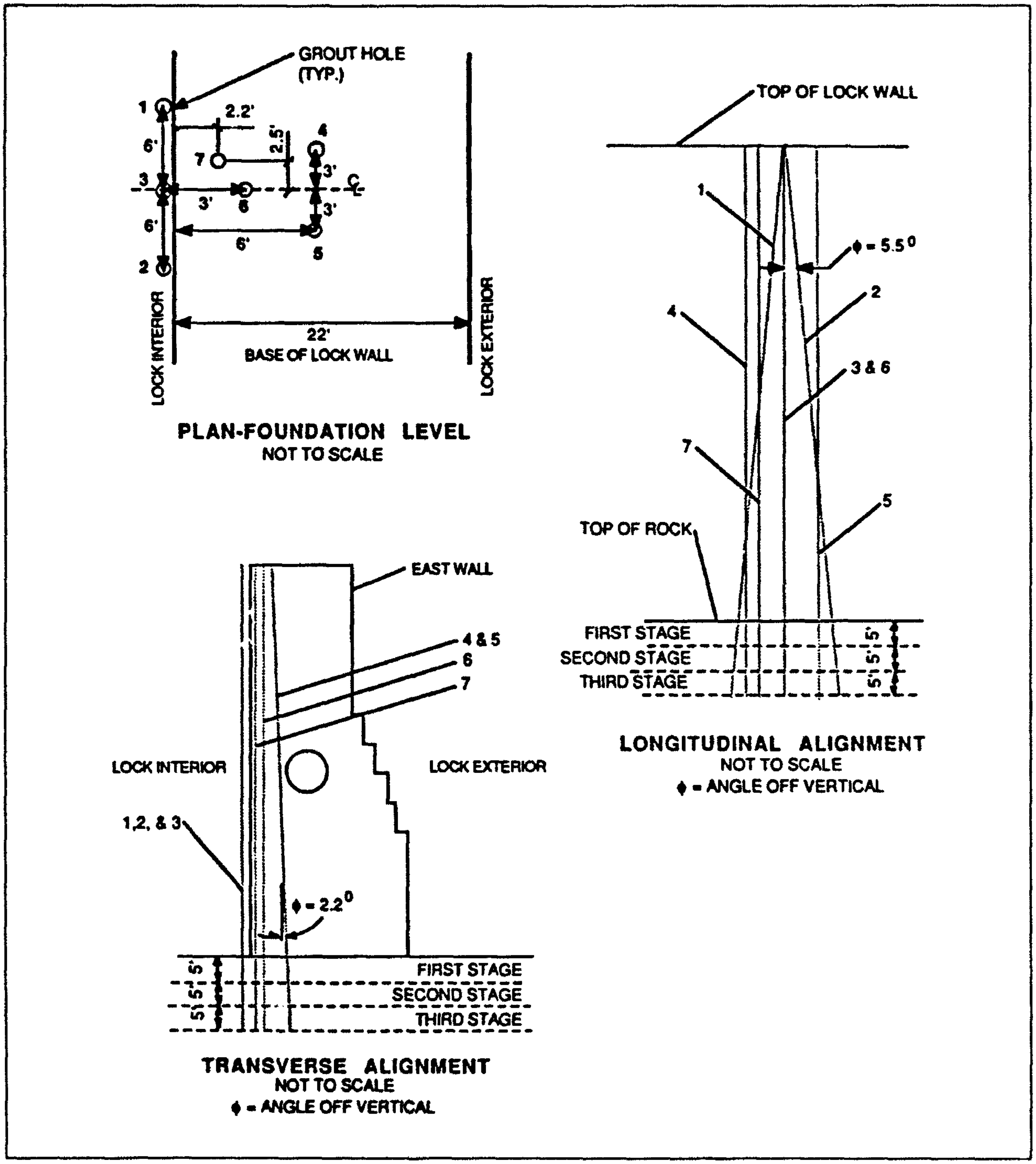

Figure 4. Test grout sections B & C

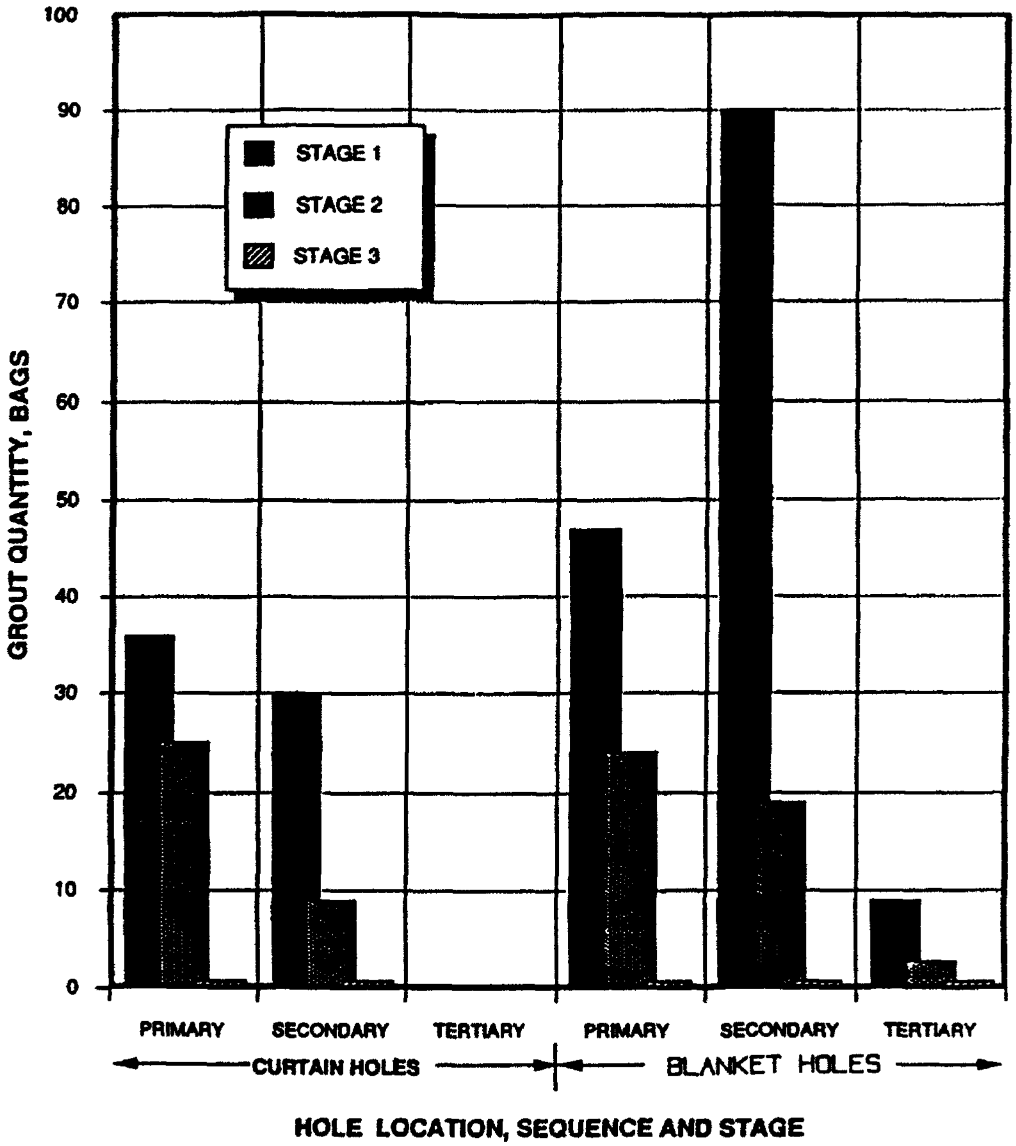

Figure 5. Average grout take versus hole sequence

allowable were obtained to adequately fill the voids and fractured bedrock. However, as a precaution the computed maximum allowable grout pressures are contained in the production grouting specifications to prevent uplift and damage to the lock walls. For the curtain holes, the maximum allowable grout pressures are 15 psig for stage 1 and 40 psig for stage 2. For the blanket holes beneath the lock walls, the maximum allowable grout pressures are 50 psig for both stages.

Grout hole spacing and number

The recommended typical grout hole grid for each 24-foot length of lock wall is shown in Figure 6. This grid provides primary holes spaced at 12 feet with secondary holes being split at a spacing of 6 feet. If needed, tertiary holes will be further split spaced at 4 to 5 feet from primary and secondary grout holes. Depending upon rock conditions, additional quaternary holes may be drilled and grouted at a spacing to be determined. In addition, 4-inch diameter check holes will be cored and video taped to determine how successfully the nearby grout injection holes are filling the voids. Initially, 1 check hole is planned for every 50-foot section of the lock wall. Additional check holes may be added depending upon field conditions. It is estimated that approximately 1,200 grout injection and check holes will be drilled depending upon the rock foundation conditions.

Stability Problems and Solutions

Results of stability analyses

Results of recent stability analyses show that the lock chamber walls of the Black Rock Lock meet current Corps of Engineers overturning and sliding stability criteria as contained in ETL 1110-2-310 for the normal operating condition with and without earthquake loading. The lock chamber walls meet sliding stability criteria but do not meet overturning stability criteria for the dewatered condition. Results of the overturning stability analyses for the dewatered condition are given below in Table 1.

Table 1
Overturning Stability Analysis Results for the Dewatered Condition (Existing Conditions)

Wall	Soil Parameters Used	% Base in Compression		Max Base Pressure, ksf	
		Req'd	Actual	Allowable	Actual
West	Short Term	50	14.1	40	58.6
West	Long Term	50	8.5	40	96.8
East	Short/Long	50	12.1	40	69.5

Remedial measures considered

In order for the lock walls to meet current Corps of Engineers overturning stability criteria, remedial measures would be required. Seven different alternatives were considered. A description and cost of each alternative considered are given in the tabulation below.

Alternative	Description	Cost, $
1	Prestressed rock anchors with permanent anchor heads. Total 120 anchors with a working load of 280 kips each.	3,510,000
2	Prestressed rock anchors with temporary anchor heads (permanent anchorage in lock wall achieved by bonding). Total 120 anchors with a working load of 280 kips each.	3,380,000
3	Non-prestressed rock anchors, inclined through total height of lock wall. Total 90 anchors with an allowable load of 384 kips. Anchors would consist of 2 #18 epoxy-coated Grade 60 rebar.	2,430,000
4	Non-prestressed rock anchors, vertical and installed through back steps of lock wall. Total 106 anchors with an allowable load of 384 kips. Anchors would consist of 2 #18 epoxy-coated Grade 60 rebar.	2,250,000
5	Temporary horizontal steel pipe compression struts installed 7.5 feet below top of wall. Total 23 struts built of 26-inch diameter A53 steel pipe, hung from temporary L-shaped hangers bolted to top of lock walls.	1,460,000
6	Removal of soil behind lockwall to decrease overturning forces.	Not Estimated
7	No Action.	--

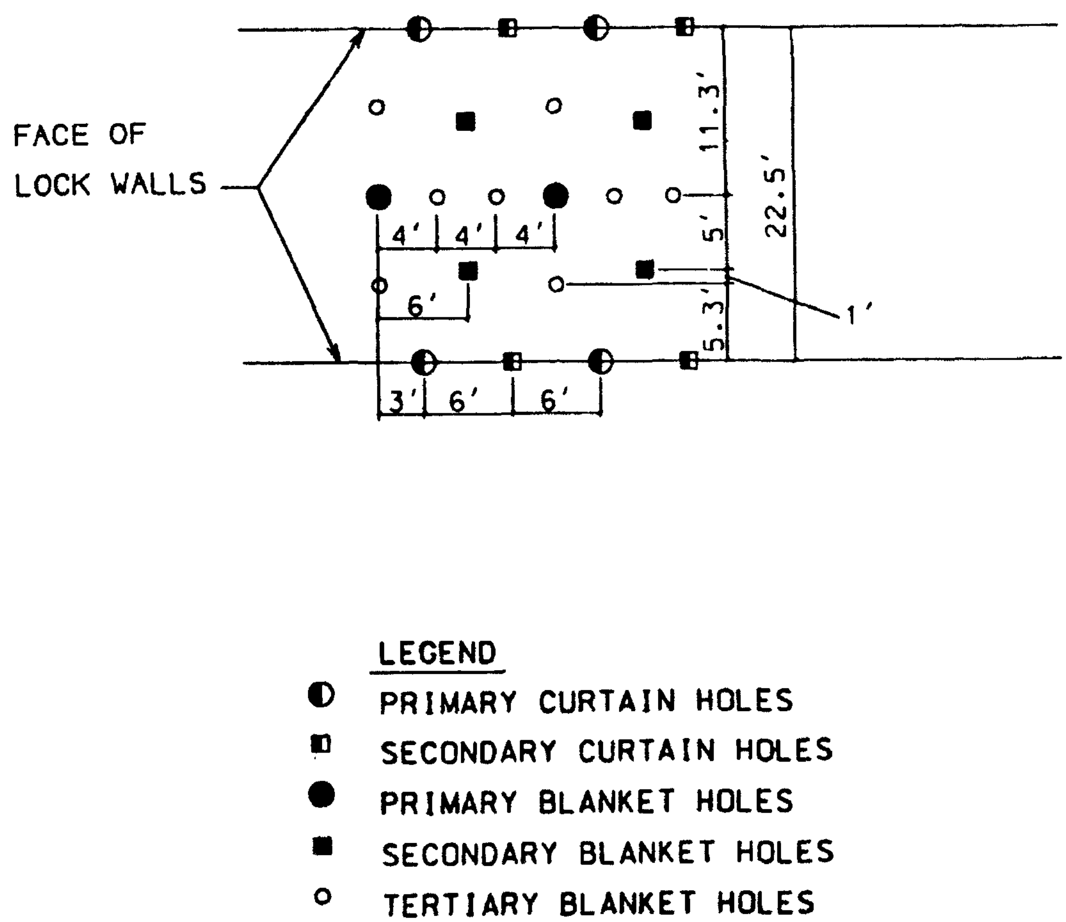

Figure 6. Typical lock wall grout hole spacing

Remedial measure selection

After consultation with North Central Division, the strut alternative (No. 5) was determined to be the most acceptable remedial measure and selected for implementation. The strut alternative had the lowest total cost, satisfied all current Corps of Engineers stability criteria, would not distress the lock walls, had been successfully used by other Corps districts, and was determined to be the most feasible of all alternatives considered. The prestressed rock anchor alternatives were rejected due to their high cost and the possibility that the high anchor forces could overstress the aging lock wall concrete. The non-prestressed rock alternatives were rejected because of their high cost and ETL 1110-2-310 recommendations against using non-prestressed rock anchors in permanent structures if other options are feasible. The alternative for removing soil behind the lock walls did not produce a significant increase in the percent of base in compression and was dropped from further consideration. The "No Action" alternative was considered not acceptable since the actual percentage of base in compression was extremely low and did not exceed the minimum 50 percent base in compression requirement, and the allowable foundation bearing pressures were exceeded.

Additional study

Since the District had recently awarded a major contract to grout the numerous voids in the rock foundation below the lock walls, it was considered possible that actual uplift pressures on the base of the lock walls would be effected by the grouting and would be different from the assumptions used in the previous stability analyses of the lock walls. The foundation grouting contract for the Black Rock Lock involves the comprehensive grouting of the rock foundation of all lock and miter gate sill monoliths. If successful, the grouting program should completely fill all voids under the lock walls and make the foundation more watertight. District engineers are currently considering modifying the foundation grouting contract to include the installation of a series of piezometers in various locations along the east and west lock walls. The piezometers would be installed in previously drilled grout and check holes to minimize installation costs. After the grouting program is completed, the newly installed piezometers would be monitored during a partial dewatering of the lock chamber. An assessment of the monitored uplift pressures would be made for the partial dewatering and used as the basis to estimate uplift pressures for a full dewatering. This would allow a reassessment of the selected remedial measure (struts) and could lead to a reduction in the required number and/or size of the struts. The estimated uplift pressures would be verified in the next actual full dewatering. It is not believed that the uplift pressures after grouting would decrease enough to completely eliminate the need for struts or other remedial measures.

Conclusion

A foundation production grouting program is currently under contract at the Black Rock Lock. This program is designed to improve the bearing capacity of the lock wall foundation, improve sliding resistance of the lock walls, and substantially decrease or entirely eliminate seepage under the lock walls by filling large cavities discovered during the 1988 subsurface exploration programs. These voids or cavities have resulted from the continuous solutioning of gypsum since lock construction. The foundation production grouting program is expected to be completed by May 1993. It is estimated that 1,200 grout injection holes will be required, most of which are drilled through portions of the lock walls. In April 1991, construction bids for this work were opened with the lowest bid of $3,553,044 coming from Hydro Group, Western Well & Pump Co., Temple, Arizona. Dewatering of the Black Rock Lock will be delayed until the foundation grouting program is completed. Steel pipe compression struts will be fabricated prior to the next scheduled dewatering of the lock chamber. The struts will be temporarily installed during each future dewatering of the lock chamber. While the pool within the lock chamber is at normal levels, the struts will be

stored on the Buffalo District reservation grounds. If the District's proposed additional study and monitoring of actual uplift pressures on the base of the lock walls is performed and yields positive results, the selected remedial measure consisting of struts may be modified.

References

Bid Abstract Black Rock Lock - Grouting Foundation Voids, Solicitation No. DACW49-91-B-0006.

Hanson Engineers, Inc., Shannon, and Wilson, Inc. 1990 (Apr). "Geotechnical Design Appendix and Test Grouting Program Major Rehabilitation, Black Rock Lock," Volumes I, II, and III, prepared for US Army Engineer District, Buffalo, Buffalo. New York.

US Army Corps of Engineers District, Buffalo. 1991 (Mar). "Black Rock Lock, Buffalo, New York, Grouting of Foundation Voids," Solicitation No. DACW49-91-B-0006, Buffalo, New York.

Evaluation and Rehabilitation of Lock Walls in the Mobile District

by

Munther N. Sahawneh[1]

Abstract

Over the past 10 years the Mobile District has discovered, either through visual observations or the PICES program, structural deficiencies at a number of its older locks. These problems were caused primarily by earth backfill pressures and saturation lines that were higher than those assumed in the design of the structures. The District has had to perform remedial work at four locks as a result of these problems at a cost approaching $3 million.

Jim Woodruff lock, which was completed in the late 1950's, had stability problems caused by a high saturation line in the fill behind the landside lock wall. The problem was discovered through the PICES program and was corrected by the installation of a backfill drain in 1981.

Holt lock, which was constructed in the 1960's, developed excessive backfill pressures in the upper gate area by the esplanade. Visual observation of a boil at the concrete/soil interface on the back side of the upper transition monolith and gradual movement of the top of the monolith toward the lock chamber triggered a full scale structural investigation. A concrete drilling program found that two monoliths just downstream of the upper gate were completely cracked into two pieces through the lock culvert. Installation of post-tensioned anchors and removal of some backfill material were required to restore the integrity of these structures. This work was completed in 1982.

Demopolis lock was completed in the early 1960's. A structural evaluation of the Demopolis lock land wall was performed because of high piezometer readings in the fill behind the lock wall. The evaluation revealed serious stability problems which resulted in the removal of 20 feet of backfill material from the upper gate area to the lower guide wall.

The Millers Ferry lock, another project completed in the 1960's, was recently found to have a problem almost identical to the one at the Holt lock. The repair will be similar to the Holt repair and should be completed in FY91.

The problems encountered at these four locks may be indicative of problems yet to be discovered with other projects constructed in the 1950's and 1960's.

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Introduction

Four locks in the Mobile District were discovered to have structural deficiencies which were caused primarily by backfill pressures and saturation lines higher than design assumptions. The high backfill pressures were caused by the presence of silts and clays in the backfill.

Two of those projects developed cracks between the culvert and the back face of the lock land wall monoliths. These two projects had no reinforcement around the culvert (designed in the 1950's).

The stability analysis indicated that these projects were unstable in their present condition. The remedial actions that took place were to reduce the elevation of the saturation line, remove or replace some of the backfill, install backfill drains, and, for the cracked monolith, to install post-tensioned anchors and grout the cracks.

Jim Woodruff Lock and Dam Project

This project is located on the Chattahoochee River, Florida. It consists of a fixed crest spillway 1,634 feet long on the right bank, a single left lock with a usable chamber dimension of 82 by 450 feet and a maximum lift of 33 feet, a gated spillway 766 feet long, a powerhouse with a switchyard and substation, and an overflow dike section 2,130 feet long on the left bank. The project was completed in 1957.

Problem

There was one basic criterion change that resulted in greater overturning forces being exerted on the structure and that is the change in the uplift criterion.[1] Uplift pressure in the original design was assumed to be effective over two-thirds of the base area. Under current criteria, the uplift pressure must act over 100 percent of the base area.

The lock land wall was designed with backfill to El. 60 feet and its stability is controlled by the saturation level in the backfill. Free-draining backfill was specified in construction, however, the presence of silt and clay in the backfill caused the saturation line to be close to El. 60 feet (about 15 feet above lower pool).

In the second PICES inspection, which was made in May 1976, a study of the design features of the lock and dam was made. This study evaluated the stability of the structures and concluded that the saturation line should be lowered about 10 feet to satisfy stability requirements.

Solution

A drainage trench was installed in June 1981. After installation, the drain flowed at about 25 gpm, and then the flow decreased after about 2 days. The before and after piezometer readings indicated a drop in the saturation line behind the lock wall after the completion of the drainage trench. The saturation line dropped to safe levels with normal tailwater. When the saturation line stabilized at about El. 50.0 feet, the problem was considered eliminated.

Holt Lock and Dam Project

This project consists of a lock and dam located about 4 miles northeast of Tuscaloosa, Alabama, on the Black Warrior River. Principal features include a lock mound and a lock on the left bank with chamber dimensions of 110 by 600 feet and provides a maximum single lift of 63.6 feet, a 680 foot long gated spillway across the river, and a 412 foot long concrete abutment section in the right bank which contains a hydropower generating plant owned and operated by the Alabama Power Company.

1 Headquarters, Department of the Army. 1964 (Apr). EM 1110-2-16-2, Washington, DC.

Problem

The problem area was the upper gate and upper transition monoliths on the land side lock wall. Construction on the lock was completed in 1966, and the top of the upper transition monolith, which is Monolith 7L, had been moving toward the chamber at a very slow rate from that time until 1977. The total movement during that period was about 1/10 inch. During the period between July 1977 and May 1980, the lock wall moved an additional 3/4 inch. An investigation was initiated in June 1980 to determine the cause of the movement.

In addition to the movement of Monolith 7L, there was also a spring noted at the concrete-fill interface behind Monolith 7L and abnormal settlement and cracking of the concrete esplanade behind Monolith 7L.

A variety of instrumentation, including alignment and settlement plugs, heave points, piezometers, and slope indicators were installed in and around Monolith 7L in an effort to determine the cause of the movement. Data obtained from borings and these instruments showed the following—a gradual shifting of the fill behind Monolith 7L toward the chamber, a piezometric head at the backfill to rock foundation interface was slightly below the upper pool level of El. 187 feet; the spring mentioned earlier behind Monolith 7L is at El. 181 feet. Horizontal and vertical readings taken on the alignment plugs showed the monolith to be tilting in toward the chamber and rotating about an axis along the base about 7 feet in from the toe at foundation grade. If this alignment data were correct, it would have meant that either the heel of Monolith 7L had lifted off the foundation or the monolith had cracked at some location, possibly the toe or the heel. Borings drilled through the heel showed that the monolith had not separated from the foundation nor did we find any cracking at the location of the borings. However, a crack in the monolith was found at a later time.

A settlement analysis indicated that foundation settlement could not have caused the movement that occurred and a structural analysis proved that elastic structural deflections were much too small to be of any significance.

The tilting movement of Monolith 7L was believed to be due to excessive earth pressures caused by saturation of the clay backfill material.

Based on the above information, it was decided to install relief wells in the fill behind Monolith 7L to relieve the head at the fill-rock interface or install shear keys in the monolith joints on either side of Monoliths 7L and 8L.

It was believed that these two measures would relieve the excessive backfill pressure and stop the movement of Monolith 7L. Installation of the relief wells and shear keys was completed in October 1980, but, in the following January-February 1981, an increased flow was observed in the spring at the fill-concrete interface. Fearing that this water may have been coming from a ruptured waterstop, it was then decided to install new waterstops in the 6L-7L and 7L-8L joints. The waterstop installation was completed in April 1981.

During drilling operations for installing these waterstops, a crack was discovered in Monolith 6L. Further exploratory drilling showed that the crack extended horizontally all the way across Monolith 6L at about the same elevation.

Solution

A stability analysis of the portion of Monolith 6L above the crack showed it to be outside design limits using lateral earth pressure coefficients of 1.0 and 0.8. Engineers investigated several alternatives for permanently stabilizing this section of Monolith 6L and came up with post-tensioned anchors as the most effective means. Six, 600-kip anchors would be required to stabilize Monolith 6L. It was also decided at this time that a number of anchors would be installed in Monolith 7L at a minimal cost since we had to mobilize to install anchors in Monolith 6L. The additional cost would be justified by the improved stability of Monolith 7L considering the high backfill pressure and the observed past movement.

The anchors specified were 0.6-inch diameter, seven-wire, stress-relieved strand conforming to ASTM A416-80, Grade 70. There were 17 strands per anchor, and the total length of the anchors was to be 95 feet in Monolith 6L and about 140 feet in Monolith 7L. The anchors in Monolith 6L would tie the two cracked pieces together, while those in Monolith 7L would extend on into the foundation to provide additional stability. The anchors were to be placed in a 6.5-inch diameter hole with a 30-foot bond length which would be grouted and allowed to set for 10 days before stressing. The design load on the anchors was 0.6 fy (598 kips for 17 strands). Each anchor would be tested to 0.785 fy and then locked off at 0.7 fy. During drilling operations for the installation of the anchors in Monolith 7L, a crack was indeed located in that monolith between the culvert and the back face at about El. 120. Further drilling revealed that the crack ran the entire length of the monolith at about a 45-degree angle from the top, landside corner of the culvert to the backfill face. The crack was believed to be open about 3/8 inch, as evidenced by the rapid drawdown of water in the anchor holes when water was passed through the culvert. Also, an amount of muddy water was observed, and the quantity of water coming out of the top of the hold increased drastically (300-500 gpm) when the drill bit reached the crack. The crack location and width was later verified by divers.

A stability analysis was then made on the cracked portion of Monolith 7L to determine what remedial action would have to be taken so that the lock could be dewatered as scheduled. It was determined that ten 667-kip anchors plus the shear keys and relief wells already in place would be a minimum requirement for dewatering the lock. Subsequently, the contract was modified to include the additional anchors for Monolith 7L.

The anchors specified for Monolith 7L were the same as for Monolith 6L except that there were 19 strands per anchor and the bond length was cut to 20 feet to keep it below the crack. The anchor installation procedure consisted of cutting a 2-foot square by 1-foot, 8-inch deep recess in the top of the lock wall where the anchor head would be recessed, and drilling a 6-1/2-inch diameter hole to the proper elevation. A 2-inch thick steel base plate was then set perpendicular to the hole and a high strength (5,000 psi) concrete pad placed around it.

The anchors were fabricated onsite by Government Forces and then placed in the holes by use of a road crane with a long boom.

The anchor bond length was grouted and, after both the first stage grout and the concrete bearing pad reached their design strength, the anchor was stressed using a 500-ton jack. The anchor hole was then grouted up to the top, and, as a final step, the excess strand was cut off and the recesses were filled with concrete flush with the top of the lock wall which has concrete strength of 3,000 psi.

Permanent stability of Monolith 7L was achieved by the anchors plus the removal of 25 feet of backfill material from behind Monolith 7L. This also required removal of part of the concrete esplanade. A permanent solution also required that the crack in Monolith 7L be grouted to stop the transfer of water between the culvert and backfill and to restore some continuity between the cracked sections. All the repairs were completed in December 1981.

We believe that the movement of Monolith 7L has been stopped. Since repairs were made in 1981, there has not been any significant movement in either the backfill material or the concrete structures.

Demopolis Lock and Dam Project

This project is located on the Tombigbee River, Alabama. It was completed in 1955. Principal features include an earth dike on the right bank, a 1,485 foot long fixed crest spillway across the river channel, a lock with chamber dimensions of 110 feet by 600 feet and a lift of 40 feet, a lock mound on the left

bank, and an earth dike across the left overbank to high ground.

Problem

Piezometers installed in 1980 behind the left lock wall indicated a saturation line higher than that used in the original design. A stability analysis of the lock wall, presented in the Demopolis PICES Report No. 5 dated 27 August 1987, showed the lock wall to be outside the current COE criteria for overturning. Subsequent to the above, additional uplift cells were installed in Monolith 8L that indicated uplift pressures higher than the original design assumptions. The soil backfill coefficients used in the original design were also reevaluated and increased based on soil classifications made when the piezometers were installed. New stability analyses were run on the land wall in February and March 1989 with results indicating that remedial action should be taken to correct the problem.

Solution

There were two basic criteria changes that resulted in greater overturning forces being exerted on the structures: the uplift assumptions and the location of the resultant of lateral earth pressures. Uplift pressure in the original design was assumed to be effective over two-thirds of the base area. Under current criteria, the uplift pressure must act over 100 percent of the area upon which it impinges. For lock walls in rock foundations, the original design used 0.33 H above the base to locate the resultant of lateral earth pressure and the current criteria uses 0.38 H.

Logs of borings taken in 1980 indicate that the backfill actually consists of clays underlain by clayey and silty sands.

Stability analyses were run on the lock land wall using the actual saturation lines and backfill coefficients that were indicated upon installation of the piezometers showed the lock wall as being unstable.

Additional stability analyses were done to determine the extent of backfill excavation required to bring the wall within the current criteria. The results of these calculations indicated that 20 feet of backfill material must be removed and the saturation line must be brought down an additional 10 feet below the fill by installing a subsurface drainage system. This action would bring the wall back within the current COE criteria for stability against overturning.

The work which was done with hired labor forces consists of removing 20 feet of backfill material from behind the wall from the upper transition monolith to the slope near the end of the lower guide wall. An existing concrete esplanade was removed and the lower lock control booth and some electrical and hydraulic lines located on and in the esplanade at the lower gate and stoplog area were relocated. Also, a collector drain system was installed in the backfill and all excavated slopes will be protected with riprap.

The left wall esplanade was used for access to the lower gate and as a staging area for lock maintenance activities. Since the esplanade area will be lost to the excavation, some type of replacement of this area and access to the lower gate will be required. As removal of the backfill material is the first order of work and, due to the lock wall stability problems, it was proposed to do the work in two phases. The first phase would remove the material from behind the lock wall and construct the collector drain system and stone protection. The second phase would return the lock to its permanent operating condition. The extent and kind of replacement of the esplanade facilities required additional engineering studies and work will be completed in FY 91. The first phase is ongoing and will be accomplished in FY 91.

Miller's Ferry Lock and Dam

This project is located on the Alabama River, Alabama. It consists of an earth dike on the right bank, gated spillway with seventeen

50-foot gates in the river channel, a single left lock with usable chamber dimensions of 84 by 600 feet and a maximum lift of 45.0 feet, a lock mound on the left bank, an earth dike extending downstream paralleling the lock to the powerhouse intake structure, a powerhouse, and an earth dike extending to high ground on the left bank. Construction began in 1963, and the lock was completed in 1969.

Problem

In April 1990, a boil was discovered at concrete backfill interface behind Monolith 6L that was flowing clear water at about 2 to 3 gallons per minute. A review of the PICES documents showed that the top of lock Monoliths 4L and 5L had moved approximately 1/2 inch toward the lock chamber over a period of about 8 to 10 years. A week later, the hole was inspected again. It was 3 feet wide, about 8 feet long, and flowing an estimated 15-20 gallons per minute of clear water. It was determined at that time that there was a clear opening between the water in the hole and in the lock culvert upstream of the upper tainter valve. This was determined by observing the water in the hole while the lock chamber was being filled. An investigation to determine the exact cause and extent of the damage was started. Since these symptoms were very similar to those we had previously encountered at Holt, we suspected that this monolith had also cracked in two as Holt had done. A drilling crew was sent to the site to determine if there was a crack, as suspected, in the lock wall, as suspected, and to obtain soil samples from the esplanade. Eight holes were drilled into the lock wall. Each hole had intercepted an apparent crack in the vicinity of the upper outside corner of the culvert. Two holes were drilled in the backfill to obtain soil samples. A remote control submarine video camera was obtained to survey the inside of the culvert and the monolith joints in the lock chamber. The lock was monitored weekly for progressive movement for a period of 4 months while structural analysis and design work were completed and arrangements made to start repairs. There was no indication of the danger of sudden failure nor was there any indication of piping of material out through the hole behind Monolith 6L.

Solution

After the remote video camera revealed the crack extending along the outside corner of the culvert, it was concluded that (1) Monoliths 4L and 5L are broken in the area outside and above the lock culvert, (2) the ceiling/wall intersection is at an angle greater than 45 deg to the sloping outer surfaces of the monoliths, and (3) the greatest water loss is occurring from the 4L-5L joint.

Approximately 2 months after the discovery of the hole behind the lock wall, material was observed being piped from the esplanade area in the vicinity of the crack. Approximately 8 cubic yards of sand gravel were dumped into the hole at the toe of the slope. After placement of this material, the leakage flow cleared up.

The remedial action chosen was to install tendon anchors across the crack and to remove about 10-15 feet of backfill from behind Monoliths 4L and 5L. This required holes to be drilled through the lock face at two levels (Figure 1) and 9 tendon anchors, each composed of 12- 0.6-inch diameter strands, be installed. These anchors then were tensioned to the design stress (approximately 430k) to stabilize the upper portion of the monolith. It was determined that, in order to provide the stable platform needed to drill the anchor holes through the lock face, a suspended scaffold would be needed to support the drill rig. This scaffold has been designed by Engineering personnel and constructed by Operation at the lock site.

Water leakage through the crack will be stopped by first sealing and grouting the crack at the culvert face using divers. Then the remainder of the crack will be grouted from above through holes drilled through the esplanade at about 5 feet on centers.

This work will be accomplished in FY 91. The approximate cost of the entire remedial

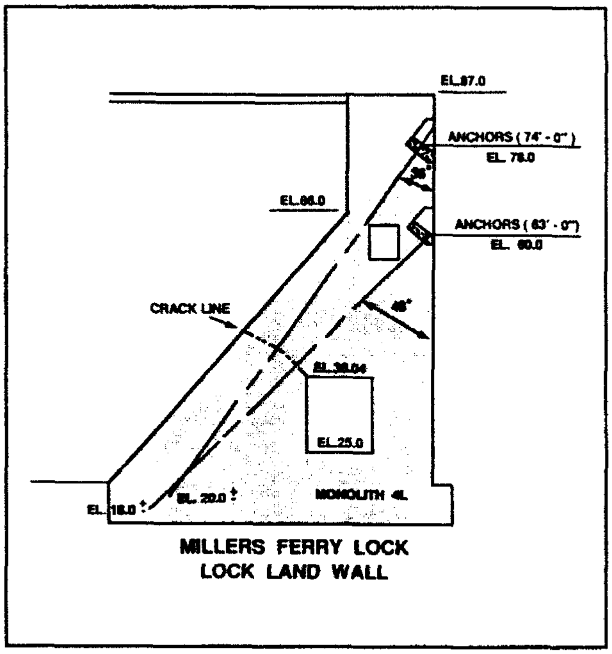

Figure 1. A side view of the crack in Monoliths 4L and 5L. This illustration shows the location of the crack and the location and dimensions of the anchors

effort was estimated at $473,000, including $346,000 for anchors, drilling and installation, $40,000 for crack seal grouting, and $87,000 for esplanade removal and fill excavation.

Conclusion

All of the problems at these projects seemed to have been caused by the fact that the random pervious fill that was called for and what was placed during construction was actually not very pervious. It had a lot of clay and fine silty material in it. It held water, causing the saturation line to rise, and the backfill coefficients used in the design for horizontal earth pressures was overly optimistic and turned out to be too low.

The two projects (Millers Ferry and Holt) which developed cracks between the culvert and the back face of the lock land wall monoliths had no reinforcement around the culvert.

The relative movement surveys at the Holt project, where the repairs have been completed, show that the tilting of the affected monoliths has been stopped, and there has not been any significant movement noted in the concrete structure.

The deficiencies that were noted at these old locks may be an indicator of problems yet to be discovered at other projects constructed in the 1950's.

Finite Element Study of Cracks in Dam Piers at David D. Terry Lock and Dam

by

Haskell E. Wright, Jr., PE[1]

Abstract

During a periodic dam safety inspection performed in December 1988, cracking was observed in the side of dam piers near the downstream water surface of 7 of the 18 piers at David D. Terry Lock & Dam (approximately 6 miles east of Little Rock, AR). The observed cracks ranged in width from hairline to 1/8 in. and continued through the thickness of the piers. Since the cracks were structurally significant, a finite element analysis modeling crack propogation was performed to evaluate dam safety and to determine possible causes of the cracking. This paper presents the results of the analyses, conclusions reached, assumptions made, and a description of the method of analysis used.

Introduction

This paper discusses finite element analyses performed to evaluate the significance and cause of cracking through the thickness of dam piers at David D. Terry Lock & Dam. The analysis included posttensioning anchorage forces, hydrostatic loadings from tainter gate trunnions, and thermal gradient loadings due to ambient temperature differentials. Cracks were traced by a "double-noding" method and compared to actual crack patterns. A commonly used program in frame analysis was utilized for the finite element analysis (STAAD-III/ISDS (Research Engineer, Inc. 1990)). The advantages and disadvantages of this particular program for performing the analsyis is also discussed.

Description of Project and Piers

The McClellan-Kerr Arkansas River System was completed in 1969 and consists of 12 locks and dams along the Arkansas River which allow navigation to the Mississippi River from headwaters in Oklahoma. The typical lock and dam consists of dam piers with tainter gates for flow control adjacent to a lock with miter gates and culvert system. The dam at David D. Terry Lock & Dam is 1,090 ft wide and consists of 16 gate bay monoliths founded on driven concrete piles (Figure 1). The piers are 10-ft-thick concrete monoliths poured in 10-ft lifts. The configuration and reinforcing of the typical pier monolith is shown in Figure 1. As-built drawings show an area of higher strength concrete (f'c = 5,000 psi) between elevations of 216 and 236 ft downstream of the tainter gates within the trunnion anchorage precompression zone.

Description of Cracking

Diagonal cracking was observed in the sides of dam piers near a tailwater surface elevation of 213 ft during a periodic inspection in 1988.

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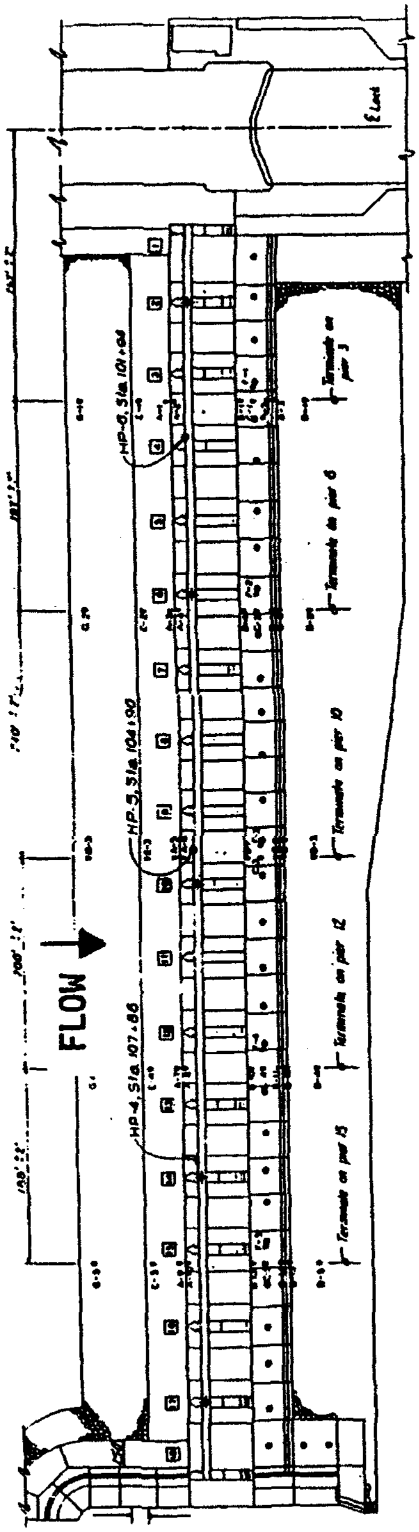
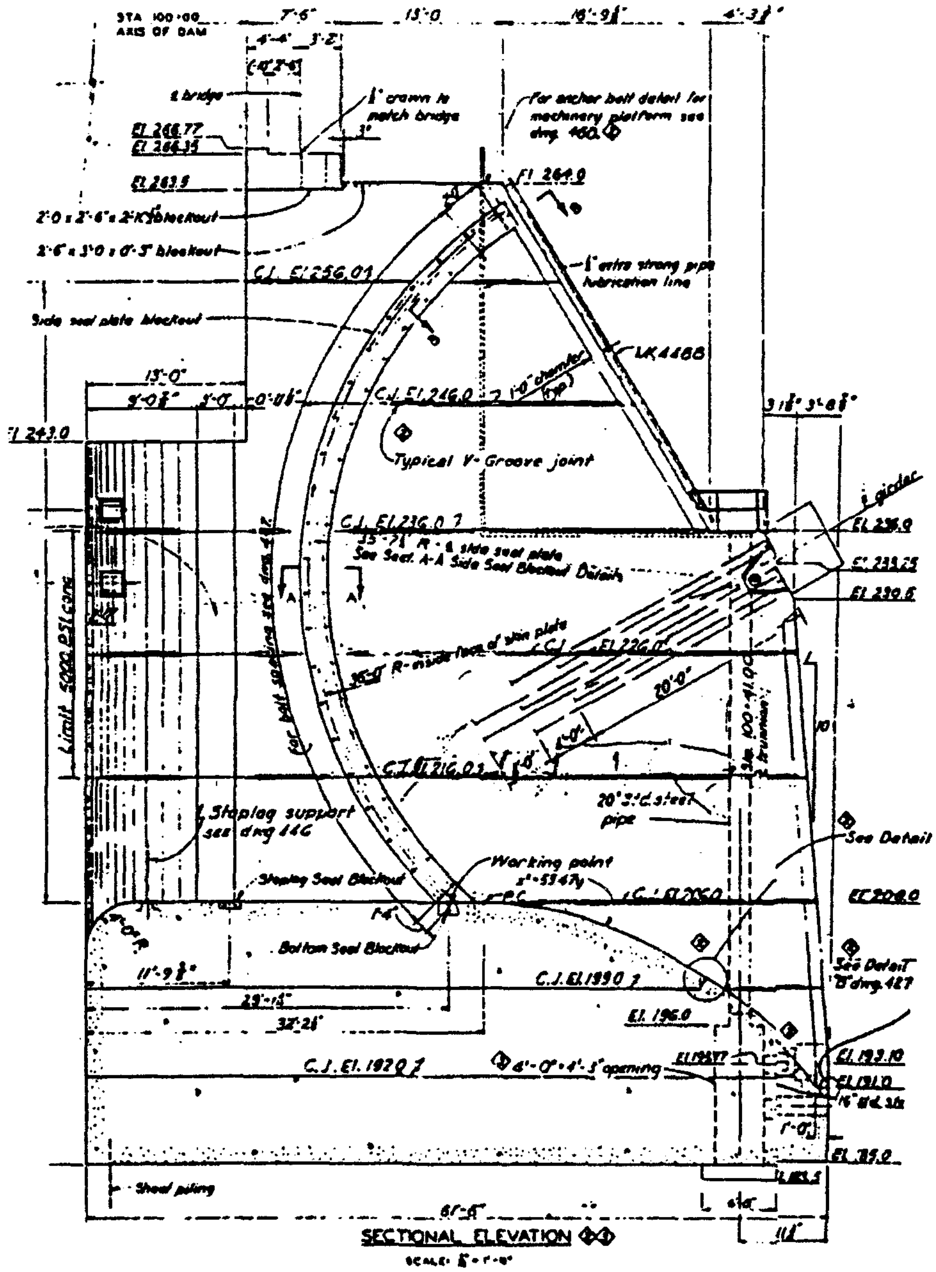


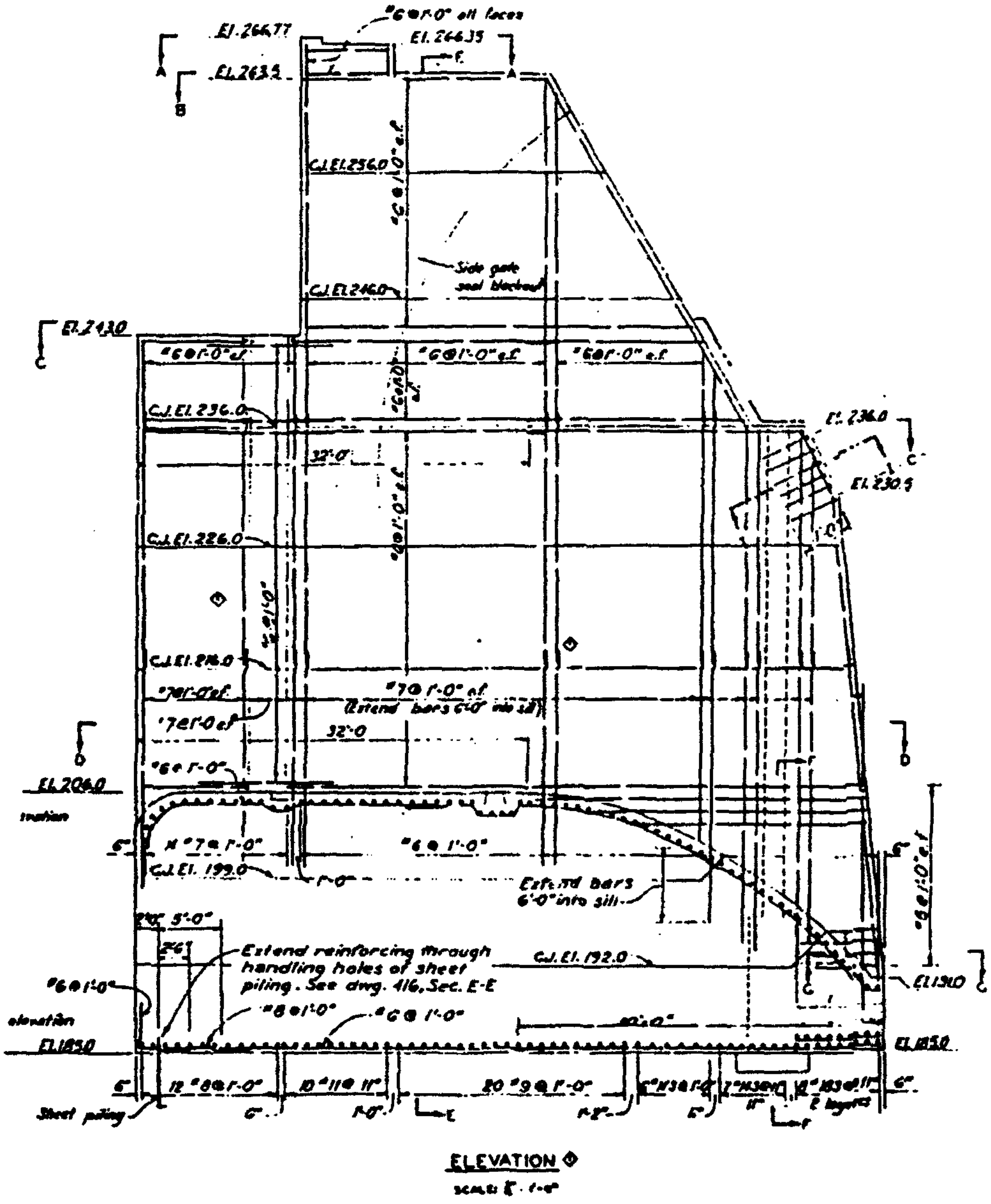
PLAN OF DAM

Figure 1. Dam pier plan and elevation (Sheet 1 of 3)



PIER ELEVATION

Figure 1. (Sheet 2 of 3)



PIER ELEVATION

Figure 1. (Sheet 3 of 3)

This cracking appeared to terminate just below the area of high-strength concrete at el 216 ft. An underwater inspection by divers was performed on dam piers 2, 4, 13, 14, 15, 16, and 17 to determine the extent of the cracking below the waterline. This cracking is shown in Figure 2. The cracking in piers 2, 4, 13, 14, 15, 16, and 17 can be described as ranging in opening width from 1/8 in. to hairline and running diagonally from a point 10 to 14 ft upstream of the downstream face of the pier at el 216 ft pour joint to the ogee near the downstream face. The angle of the crack varies slightly from pier to pier as well as the width of the cracking. The cracks appear to penetrate the full thickness of the 10-ft-thick piers. Also, cracking was observed running parallel to the ogee near the tainter gate on pier 2. Only small hairline cracking was observed above the el 216 ft pour joint near the posttensioning anchors. The sill/pier interface area upstream of the tainter gate was not inspected. No

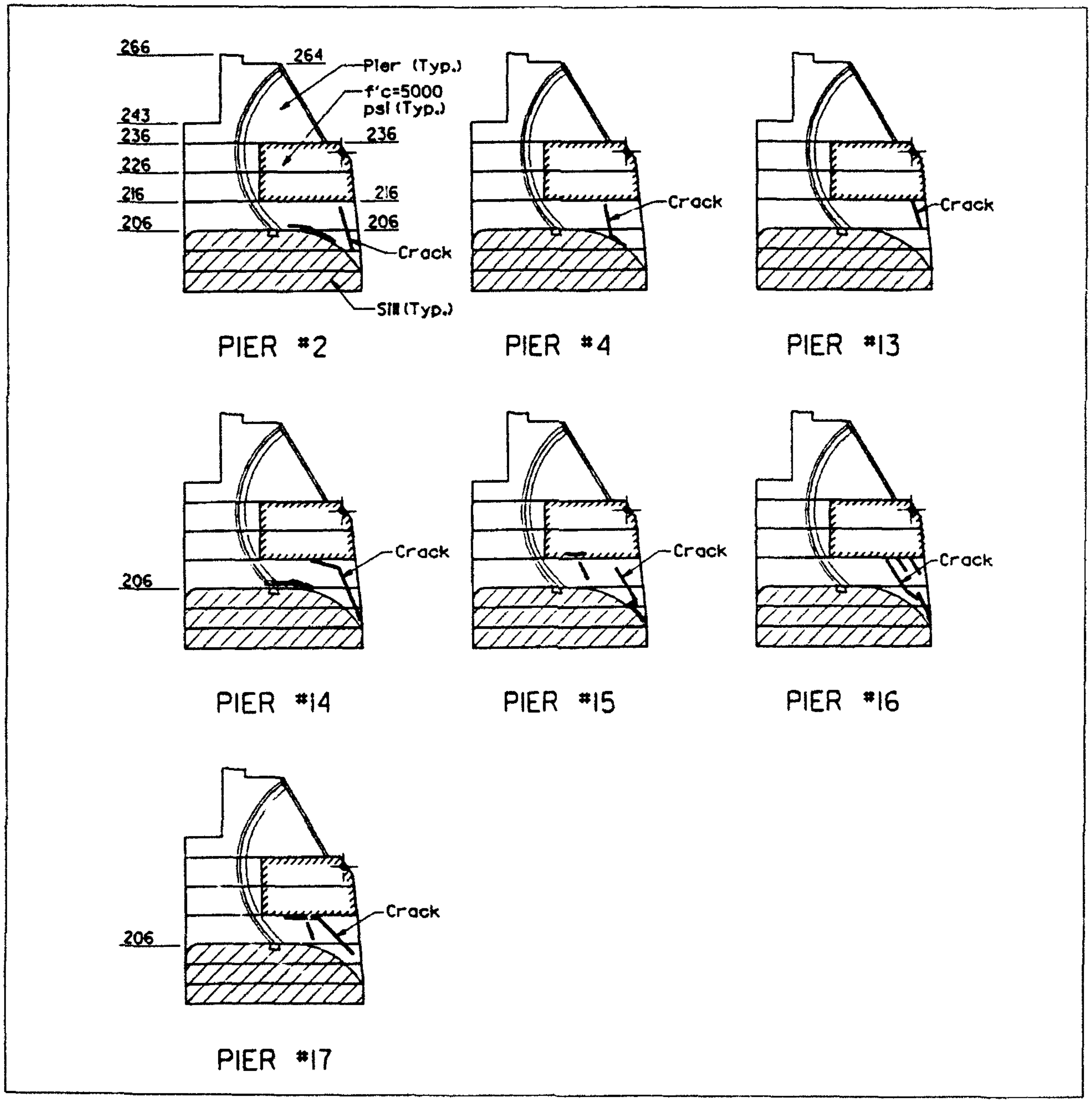


Figure 2. Crack locations in dam piers

cracking was visible above the waterline at el 213 ft upstream of the tainter gates.

Historical Data Review and Material Tests

As background data, previous periodic and annual inspections were reviewed as well as information on potential seismic events in the area. The cracking observed in the 1988 inspection was not mentioned in any previous report. Seismic historical records did not indicate recent seismic activity sufficiently close to the site to cause measurable structural damage. Settlement and lateral movement measurements recorded in periodic reports from 1978 to 1988 were evaluated to see if any patterns would explain the cracking. No significant pattern of movement was observed which would explain the cracking observed. Barge impact records were obtained and evaluated. Recorded barge impacts did not appear to cause damage consistent with the crack patterns observed. To validate material properties to be used in the analyses, concrete core samples were obtained from piers 17 and 5 in January 1990. Compressive strengths for an average of three test locations in pier 17 exceeded 5,000 psi (3,000 psi assumed in the original design analysis) and thus low concrete strength did not appear to be a factor in the cracking.

Initial Evaluations

To assist in making an initial evaluation of the causes for the cracking in the dam piers, inspections were performed on all remaining locks and dams in this river system to determine if the cracking pattern observed was common to other dam piers. An end pier at Lock and Dam 3 had similar cracking. However, the remaining dam piers at L&D 3 and at other locks and dam did not show this cracking behavior. The percentage of reinforcing in the cracking area of the Lock and Dam 6 pier was compared with reinforcing in other dam piers. Percentage of reinforcing in the region of the posttensioning anchorage in L&D 6 was slightly lower than in other dam piers. Also, the detail showing embedment length of the horizontal reinforcing in the sill (#6 at 12 in. on center) was unclear on the L&D 6 drawing and the in-place horizontal bars may have inadequate development length at the sill/pier interface.

Based on the inspections performed, possible causes of the cracking were identified as: (1) deficient design for hydrostatic design loads; (2) occurrence of unusual impact loadings above design level from barges; (3) inadequate design for unidentified stress concentrations from the posttensioning anchorages; (4) inadequate design for ambient or adiabatic thermal stresses; and (5) excessive stresses caused by shrinkage, creep, or construction sequencing. Due to concern over possible impacts on dam safety caused by the observed cracking in the dam piers, plane stress finite element analyses were performed to obtain insight into elastic behavior and potential crack propogation.

Analysis Description

Computer program

The computer program used to perform the analysis was STAAD-III/ISDS, revision 12.1, issued by Research Engineers, Inc. (1990). This program was selected due to availibility, user familiarity, and low cost. STAAD-III/ ISDS is extensively used for building applications but is not commonly used for finite element analsysis of hydraulic structures. Limitations of the program (especially in graphic postprocessing of element stresses) were troublesome but did not compromise the basic objectives of the analyses. During the time the analysis was being performed, the program was upgraded to include thermal finite element loadings. This allowed use of the program for thermal analysis. The model utilized plane stress quadrilateral and triangular elements. The program does not have capability to perform a NISA time history type analysis as described in ETL 1110-2-324 (Headquarters, Department of the Army, 1990). The program was easy to use and inexpensive compared to other time-sharing alternatives available.

Approach and objectives

Two basic sets of analyses were performed. The first set of analyses (Analysis I) addressed the capacity of the structure to resist hydrostatic and impact loadings as defined in the original design documents. This analysis was expected to reveal whether or not structure was originally underdesigned for hydrostatic load cases. The models for Analysis I assumed that the pier was fixed at the interface with the sill, and the sill was not modeled. A second set of finite element analyses (Analysis II) was performed which evaluated effects of temperature differentials between the sill and pier. Analysis II models included the sill concrete mass.

The objectives of Analysis I were to: (1) provide an estimate of strength capacity for the piers in terms of applied design loading; (2) provide an estimate of the extent and location of cracking due to various applied load levels through failure; (3) identify local stress concentrations and compare stress levels to cracking stress levels; and (4) evaluate the effect of additional posttensioning on tensile stress concentrations.

The objectives of Analysis II were to: (1) evaluate stresses caused by thermal gradients in the sill and pier structures; (2) identify local stress concentrations resulting from thermal stresses in combination with hydrostatic loadings; (3) trace cracking patterns (if present) from the thermal load cases; and (4) model the sill concrete to more correctly evaluate the boundary condition at the pier/sill interface.

Concrete tensile strength and method of cracking analysis

Tensile strength of the concrete was assumed to be $0.10 \times f'c$ (300 psi for 3,000 psi concrete) to give a conservative lower bound on potential cracking. Other references allow $1.7 \times (f'c)^{0.67}$ (Raphael 1984), which is higher than $0.10 \times f'c$. However, due to uncertainty concerning the strength of the concrete at time of cracking, the lower value of $0.10 \times f'c$ was assumed. Actual closure temperatures of the pier concrete was unknown.

Cracking behavior was modeled through an iterative procedure. Beginning with the uncracked condition, elements with stresses exceeding $0.10 \times f'c$ were "double noded" to allow relative displacement with adjacent elements. At elements with stresses exceeding $0.10 \times f'c$, the double nodes were utilized and adjacent elements were allowed to displace relative to each other. The stress levels in the elements were then checked and the section was considered stable if stresses were less than $0.10 \times f'c$. This procedure was followed in Analysis I to trace cracking patterns for incremental load levels up through three times the design loads (These load levels actually could not be reached since the trunnion block anchorage would fail prior to this level of load). In Analysis II, only a single load level was checked, and nodes were released to bring stress levels down in adjacent elements to within cracking level.

Model description

The finite element grids for Analysis I and Analysis II are shown in Figure 3. The interface between the sill and pier in Analysis I was assumed to be fixed and was modeled as a series of pinned connections. The piling supports for the sill in Analysis Model II were also modeled as pinned supports. These support conditions are shown in Figure 3.

Material description

Material properties for Analysis I and Analysis II are as follows:

Concrete Strength	
Precompressed Zone (el 216 to 236) (based on as-built drawings)	f'c = 5,000 psi
Remainder of Pier & Sill	f'c = 3,000 psi
Modulus of Elasticity of Concrete	
f'c = 3,000 psi	Ec= 3,122,000 psi
f'c = 5,000 psi	Ec= 4,030,500 psi
Unit Weight of Concrete	150 pcf
Thermal Coefficient of Concrete	7.0E-06 per deg F
Unit Weight of Water	62.5 pcf

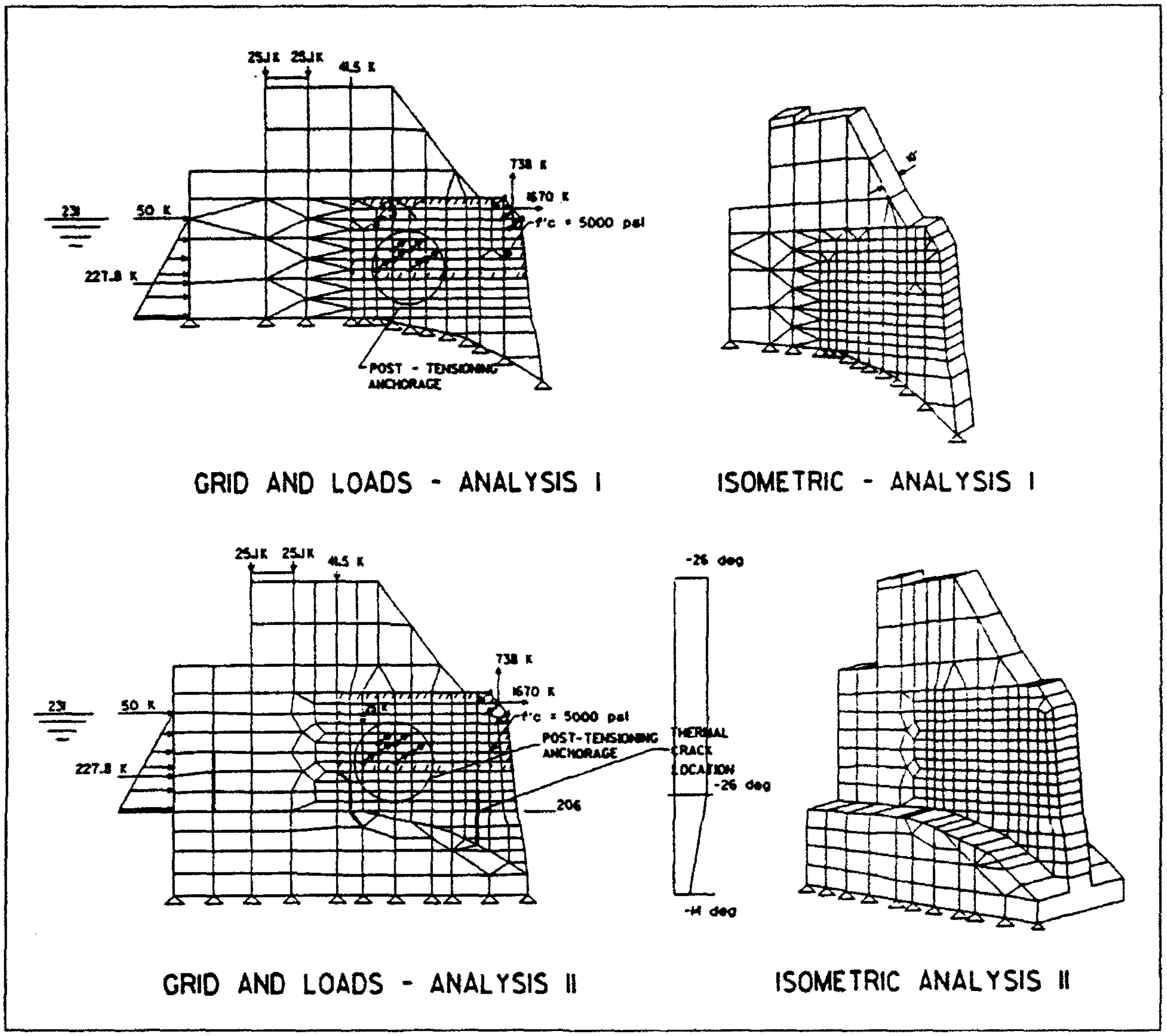


Figure 3. Finite element grids - Analyses I and II

Loading descriptions

Loads for **Analysis I** were obtained from the original design calculations for the spillway pier. Posttensioning forces used to anchor the trunnion block were included with other applied bridge and tainter gate loads. These applied loads are shown in Figure 3. The applied hydrostatic and trunnion forces were incremented up to a level of three times the initial design level to determine the ultimate capacity of the pier in terms of factored design loads.

The temperature load in **Analysis II** was due to seasonal temperature change. Thermal loading from the heat of hydration was assumed to have been completely dissipated and was not included in the analysis. For an estimated return period of 2 years, the maximum anticipated air temperature at David D. Terry is 100 °F and the minimum corresponding temperature is 9 °F, giving a 91-degree temperature differential. A temperature was assumed for the sill and pier concrete which yielded a 26-degree differential between the pier concrete above el 213 and the sill concrete at el 206. The temperature differential between the pier and sill elements at the juncture of the pier and sill was 14 °F due to seasonal temperature variation.

Analysis Results

Analysis I - Hydrostatic, impact, and dead loads

The first Analysis I model assumed an uncracked pier section. Initiation and progression of cracking was traced through a design level force of 3.0 times the design level. This force is not physically possible due to failure of the trunnion and gate but was assumed to give a capacity factor to compare to hand calculations. The analysis indicated a capacity exceeding 2.0 times the design level. Tensile stress concentrations were identified near the anchorage points of the posttensioned trunnion anchorages. A plot of maximum principal stresses resulting from the combined loads at 1.0 times the design level is shown in Figure 4. This analysis indicated that tensile forces were low and cracking should not have initiated and progressed at the pier design level forces.

A second set of analysis models in Analsyis I assumed that a crack similar to that observed at pier 16 existed at the time of initial loading. The progression of the crack was traced similarly to model 1 and indicated a capacity in excess of 2.0 times the design level force. Again, the analysis indicated that at the design level, the crack should not have progressed as observed due to applied loads. Additional Analysis I models were run to evaluate the potential beneficial effect of drilled and grouted post-tensioned rods to reduce tensile stresses near the posttensioning anchorage. The analysis showed that a large post-tensioning force would be required to significantly affect these stresses, thus making this potential repair option less desirable.

A final Analysis I model was run to evaluate the effect of a 1,000,000-lb force applied at water level (el 231) on the upstream face of the dam pier to simulate a potential barge impact. The stress increases due to this force did not coincide with the location of the cracking or the stress concentrations due to the posttensioning anchorages. The results indicated that this level barge impact force did not cause tensile stresses large enough to cause crack propogation similar to that observed.

The following can be summarized from the results of Analysis I: (1) the analyses verified that the capacity factor of safety of the pier exceeded the capacity factor of safety estimated by manual calculations; (2) design level forces do not cause tensile stresses sufficient to cause the cracking observed; (3) large posttensioned forces would be required to significantly reduce the areas of tensile stress concentrations near the posttensioning termination points; (4) local concentrations of tensile stresses occurred near the termination points of the trunnion posttensioning rods; and (5) a postulated 1,000,000-lb barge impact force on the upstream face of the pier did not cause sufficient tensile stresses to cause the observed cracking.

Analysis II - Thermal loads from ambient temperatures

Analysis II consisted of two models, an initally uncracked pier, and a pier with cracks to relieve thermal stresses. The results of the uncracked analysis model indicated tensile stress levels at the sill/pier interface above 300 psi (cracking level stress). The primary thermal effect evident from the stresses at the sill/pier interface is a lateral contraction of the pier (due to lower temperatures) together with the constraint of this contraction by the massive sill. The direct consequence of the base restraint is a system of tensile stresses in the horizontal direction at the sill\pier interface.

This induces large principal tensile stresses which would open nearly vertical cracks. The maximum principal stress contours for the combined thermal and applied loads is shown in Figure 4. The location of the stress concentrations at the sill/pier interface were near the areas of observed cracking.

Following the analysis of the uncracked pier section, a series of analyses was made to trace the cracking of the pier section due to thermal loads to a point where stress levels were below cracking. The crack was modeled by double noding and releasing the adjacent nodes at the crack location. The purpose of this model was to determine the extent of the cracking and evaluate the capacity of the pier when cracked. The results of this analysis

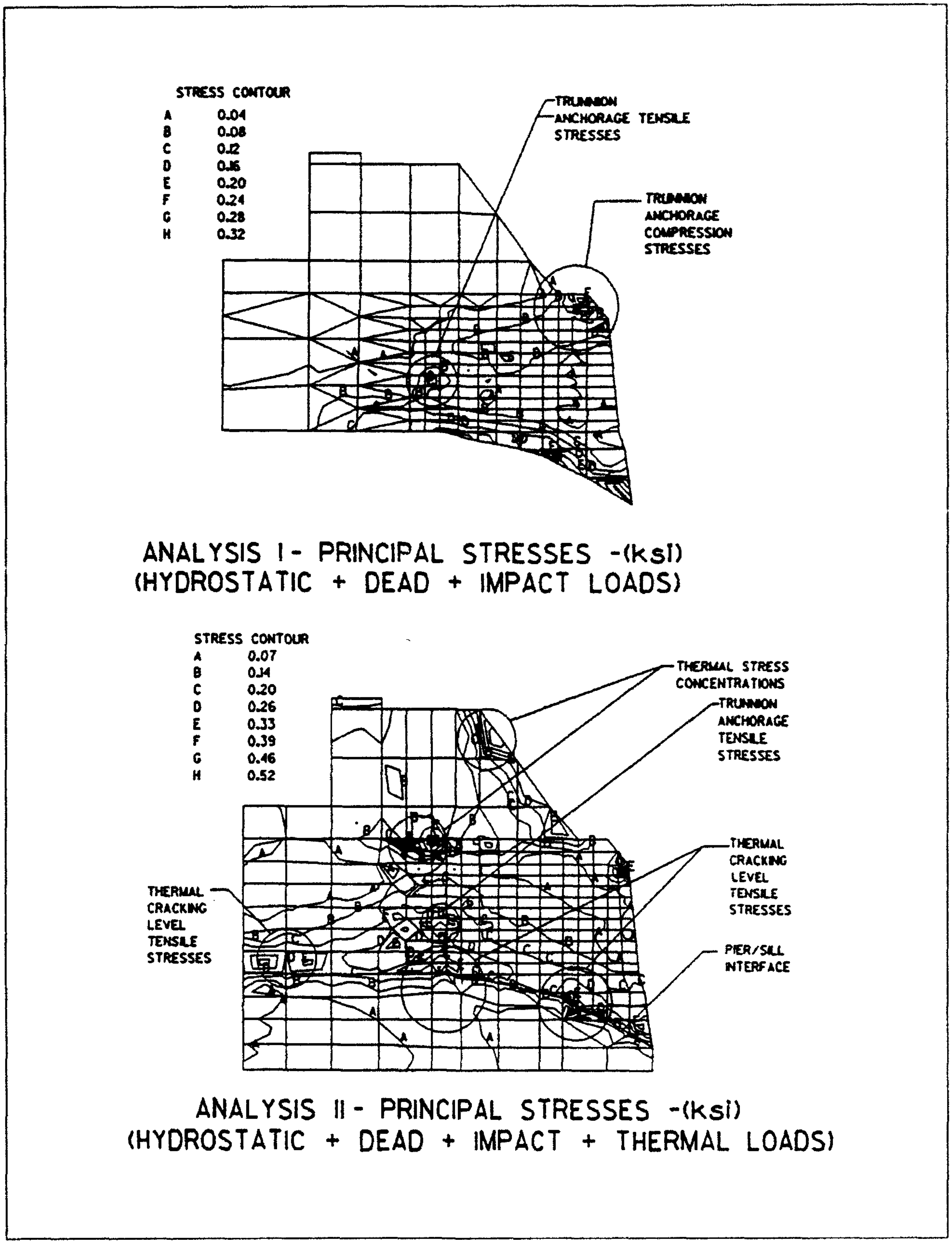


Figure 4. Maximum principal stresses - uncracked sections

indicated crack locations and sizes similar to those observed in the piers. The stress levels under full design level were generally within cracking levels and the pier was thus considered structurally safe. Maximum principal stress contours are shown in Figure 4.

Correlation of analysis results to observed cracking

The analysis indicates that the cracking patterns observed were not caused by hydrostatic overload conditions (i.e., trunnion reactions larger than design levels). However, the analysis does show a correlation between thermal tensile stress concentrations and observed crack locations. The cracking pattern may not have been caused by the trunnion forces, but the cracking did appear to originate from stress concentrations caused by thermal loadings at the sill/pier interface and progress toward the area of high tensile stress at the posttensioning termination.

Conclusions

Causes of cracking

The thermal analysis results indicated that most of the elements at the sill/pier interface experienced stress levels above 300 psi. The intensity and direction of these stresses would tend to produce vertical cracking similar to that observed in the dam piers at David D. Terry. The conclusion is that temperature gradients between the sill and pier induced high stresses which caused the observed cracking.

A secondary conclusion that can be drawn from the analysis is that the posttensioning anchorage forces cause tensile stresses in the piers. Some cracking was observed in the general area where the anchorage stresses were indicated.

Structural capacity of the cracked section

The capacity of the cracked pier to resist the applied hydrostatic loadings and thermal loads is adequate since the level of the principal tensile stresses are below cracking level in the cracked model (Analsyis II). Anaylsis I models indicated that the pier could resist additional design load levels above the initial design, even with the cracking present.

Adequacy of original design methodology

The original pier design was performed in June 1964. The analysis approach was to calculate moments and axial forces about a horizontal pier cross section at various elevations and then use a working stress analysis to obtain concrete and reinforcing stresses resulting from these forces. Moments were obtained in both the longitudinal and transverse directions (due to one gate closed and one gate open) and were applied simultaneously. Thermal stresses and stress concentrations due to the posttensioning anchorage were not evaluated in the original design analysis. Due to these omissions, the high local cracking stresses at the sill/pier interface and at the posttensioning anchorage were not identified. In summary, it is concluded that the original design procedures were inadequate to predict and design for high thermal stresses at restraint conditions and that procedures similar to that proposed in ETL 1110-2-324 (Headquarters, Department of the Army, 1990) should be followed to ensure "crack free" designs.

Cracking at other locks and dams on Arkansas River

Other piers on this project and at other Arkansas River dams do not experience the same cracks as the worst piers at Lock and Dam 6. This is possibly due to: (1) the other projects have more reinforcement in the trunnion anchorage zone; (2) concrete strengths may differ by project, by pier, or even by lift; (3) shrinkage stresses may vary by project, by pier, or by lift; (4) closure temperature and construction sequencing varied between the different projects, thus reducing the impact of thermal stresses; (5) concrete deterioration or durability may vary by project; and (6) horizontal reinforcing at the sill/pier interface has more

embedment length into the sill from the pier at other Locks which minimizes the vertical cracking.

Adequacy of computer program

STAAD-III/ISDS is an extremely easy program to use for plane stress analysis and is ideal for use on PC hardware. However, the additional time and effort required to adequately evaluate maximum principal tensile stresses and directions make the program inefficient for concrete cracking analysis. For example, the program plots stress contours only for the maximum absolute value of the principal stresses, either compression or tension. Because of this, referral must be made to printed output to determine if the contour shown in the stress plot is compression or tension. Also, the lack of higher-order elements and "cracking elements" makes it difficult to perform crack propogation analyses. The program can best be used to prepare quick elastic analyses which indicate overstressed areas. It is cumbersome to use to perform crack propogation studies, especially for thermal loads. The program does not currently have the capability to perform time-history thermal analysis.

References

Headquarters, Department of the Army. 1990 (Mar). "Special Design Provisions for Massive Concrete Structures," Engineer Technical Letter 1110-2-324, Washington, DC.

Guyon, Yvef F. 1953. *Prestressed Concrete*, Vol I, Wiley, New York, NY.

Raphael, Jerome M. 1984. "Tensile Strength of Concrete," *Journal of American Concrete Institute*, Detroit, MI.

Research Engineers, Inc. 1990. *STAAD-III/ISDS User's Manual*, Research Engineers, Inc., Marlton, NJ.

Design of Training Wall Extension Harry S. Truman Dam, Missouri

Richard A. Shanks, PE[1]

Abstract

In 1986 flooding of the Harry S. Truman Lake area resulted in large releases from the spillway at H. S. Truman Dam. A large area downstream of the existing left bank training wall was severely eroded. This erosion extended around and behind the existing training wall to an extent that additional scouring could threaten the structural integrity of the existing wall. Therefore, it was decided to raise the existing wall height and extend the wall approximately 140 ft downstream.

This paper describes the unique analysis and construction methods used for the I-wall portion of the reinforced concrete wall extension. Future erosion of up to 10 ft is anticipated on the channel side of the new wall. It was necessary to provide a 30-ft-deep, structurally sound base foundation for this wall upon which a conventionally constructed 12- to 22-ft-high wall would be built. The foundation consists of drilled, staggered 30-in.-diam holes filled with reinforced concrete. This will create an I-wall socketed in rock. The controlling load cases occurred after 10 ft of erosion on the channel side and with tailwater at floodstage or with saturated backfill.

Background

Harry S. Truman Dam is a Corps of Engineers project located in the Osage River Basin of central Missouri. The dam was completed in October of 1979 and consists of an earth/rock embankment, a concrete spillway, and a 160,000 kw power plant with six turbine-generator units. The spillway section of the dam has four 40-ft tainter gates which can supplement powerhouse releases in times of flooding. The reservoir has a multipurpose pool surface area of 55,600 acres, with flood pool at 209,300 acres.

Truman Dam has made spillway flood releases eight times since placed in full operation in 1982. In October and November of 1986, massive rainfall in the Osage River Basin caused major flood releases to be necessary from the Truman Dam spillway. Spillway releases of up to 41,300 cfs in conjunction with powerhouse releases up to 28,000 cfs (total discharge close to 70,000 cfs) continued for approximately 52 days. These releases caused major damage to the riprap, underlying rock, and fencing on the spillway side bank downstream of the concrete spillway training wall. The downstream section of this training wall tops out at elevation (el) 665.5 and is anchored to the bank with rock anchors. The damage to the rock extended landward and behind this section of the training wall. Future releases could threaten the structural integrity of this wall. An average depth of 3 to 5 ft of rock over an area of 4,000 sq ft was eroded and

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washed away. Much of this loose rock may have washed back into the stilling basin and contributed to serious erosion of the concrete base slab. In order to stop this erosion and resultant damage, it was decided to design a concrete spillway training wall extension (Figures 1, 2, and 3).

The new wall extension includes an L-wall to extend the height of the existing wall from el 665.5 to 682, an I-wall extending 25 ft landward and 140 ft downstream, and a wall to seal off the end of the existing wall. The structural design and construction of the I-wall portion of this project will be the subject of this paper.

I-Wall Preliminary Considerations

Project requirements

The requirements of this project are: (a) to provide protection to the bankline downstream of the existing spillway training wall against erosion due to future spillway flood releases, (b) to extend the height of the existing wall to el 682.0, and (c) to seal off the exposed area at the end of the existing wall, thereby protecting the existing rock anchors.

Project parameters

The elevation of the rock surface downstream of the existing training wall which was originally at el 662 has eroded to between el 660 to 657, with some depressions as low as el 653. It is anticipated that the erosion on the channel side of the new I-wall will continue in future spillway releases. Therefore, the I-wall had to be designed for erosion to continue to el 650.0.

Large wall loadings result in large overturning moments in this wall (Figure 4, Load Cases). Therefore, the I-wall must be well founded or socketed into the rock below el 650.0. It was decided to use el 630.0 as a bottom elevation for the I-wall foundation, resulting in a design depth of socket of 20 ft.

The spillway release of 1986 overtopped the downstream section of the existing training wall. Therefore, the top of the new wall at the upstream end would be set at el 682.0. The top of the I-wall would transition down to el 672.0 and continue at that elevation for its final 100 ft of length.

Time and scheduling constraints were critical. There exists a window of time from late fall through winter when the likelihood of high tailwater elevations is remote. Also, fish spawn heavily in the area of this project in the spring, and construction at this time would be restricted. The contractor must begin and complete construction in a timely manner. A severe winter would pose serious problems.

Project options

Because of the restrictive parameters and obvious construction problems, brainstorming sessions were held to develop various design options and methods. The following three were considered to have the greatest potential for success from a design and construction viewpoint:

- Use a rock trenching machine to excavate a deep trench in the rock. Place reinforced concrete in the trench and build a conventionally formed reinforced concrete wall up to the required elevation.

- Use a 40-in.-diam rock drill to drill a continuous row of holes extending the length of the I-wall, and fill them with reinforced concrete. Build a conventionally formed reinforced concrete wall over the drilled piers up to the required elevation.

- Use a 30-in.-diam rock drill to drill a series of staggered holes extending the length of the I-wall and fill them with reinforced concrete. Build a conventionally formed reinforced concrete wall over the drilled piers up to the required elevation.

Preliminary analysis and design revealed that any of the three methods would be

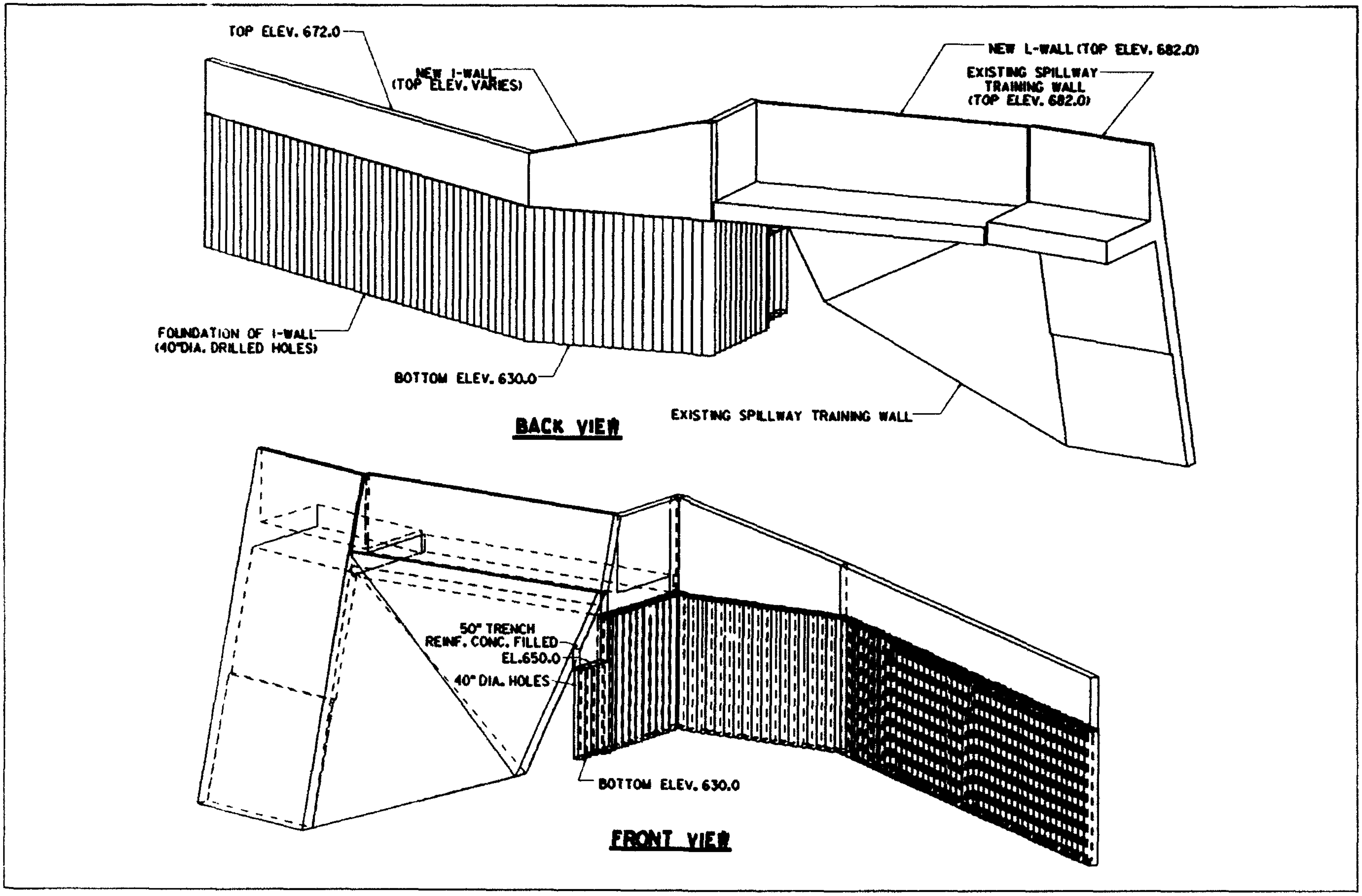


Figure 1. ISO views of I-Wall and L-Wall

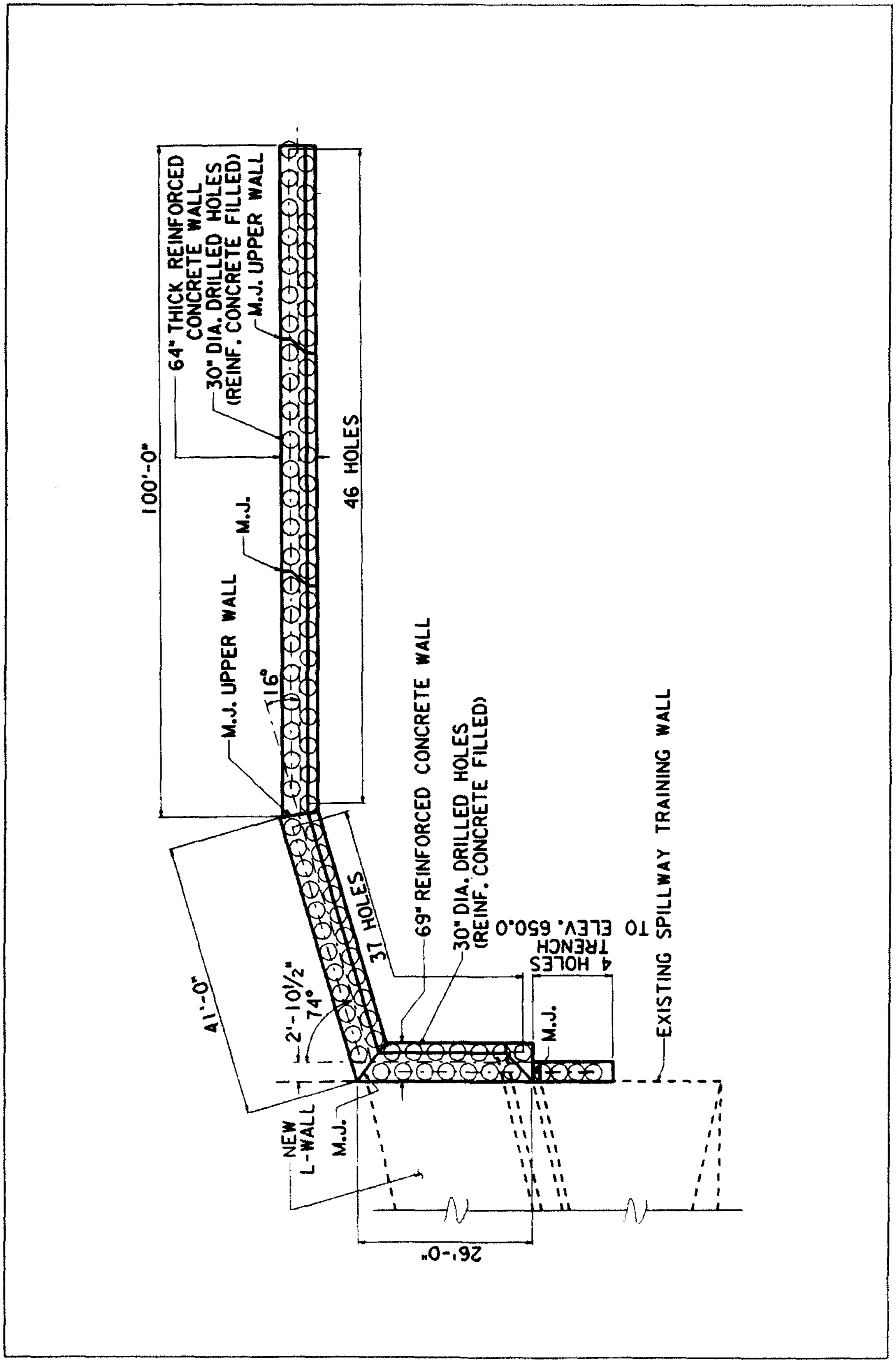


Figure 2. I-Wall plan

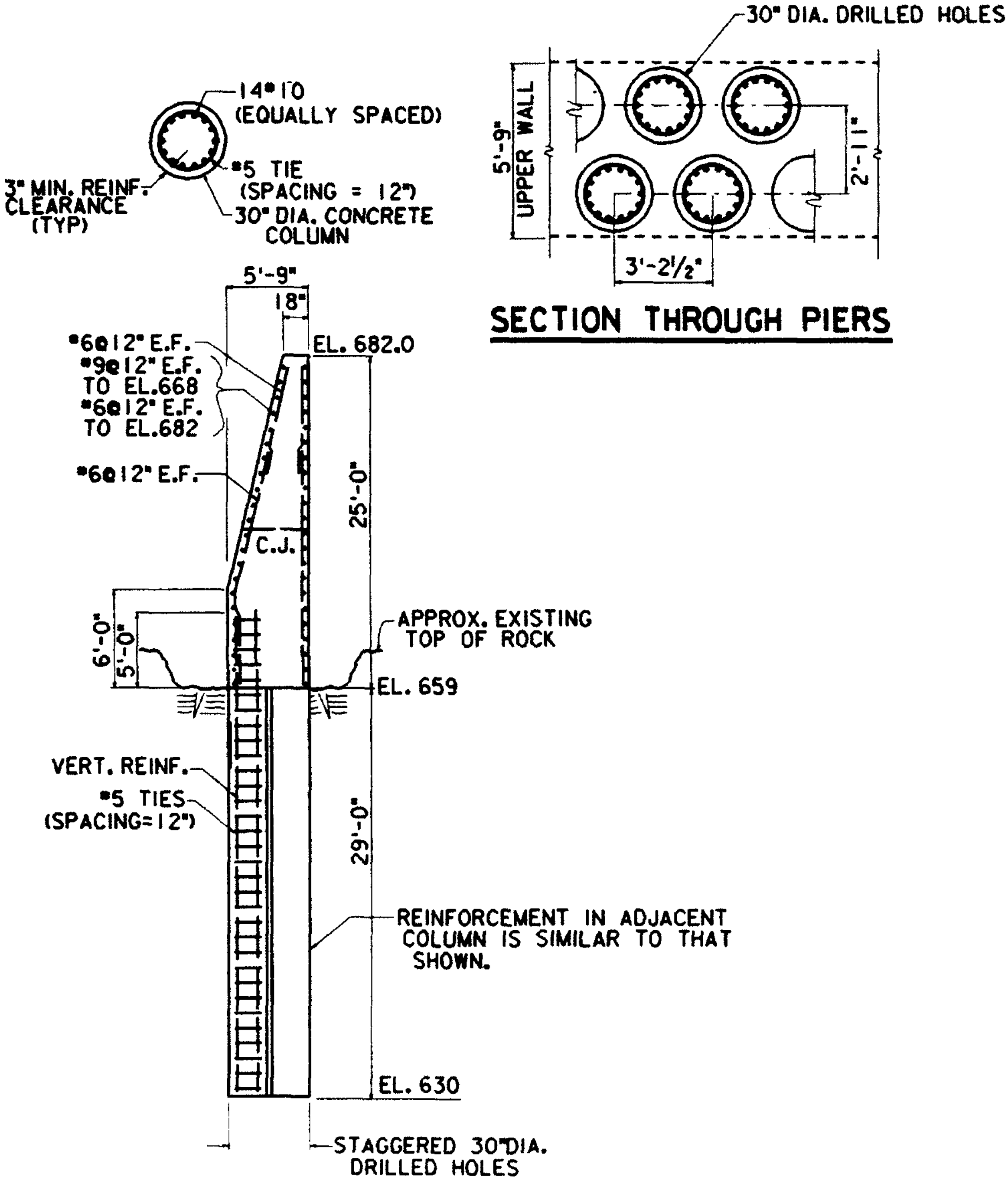
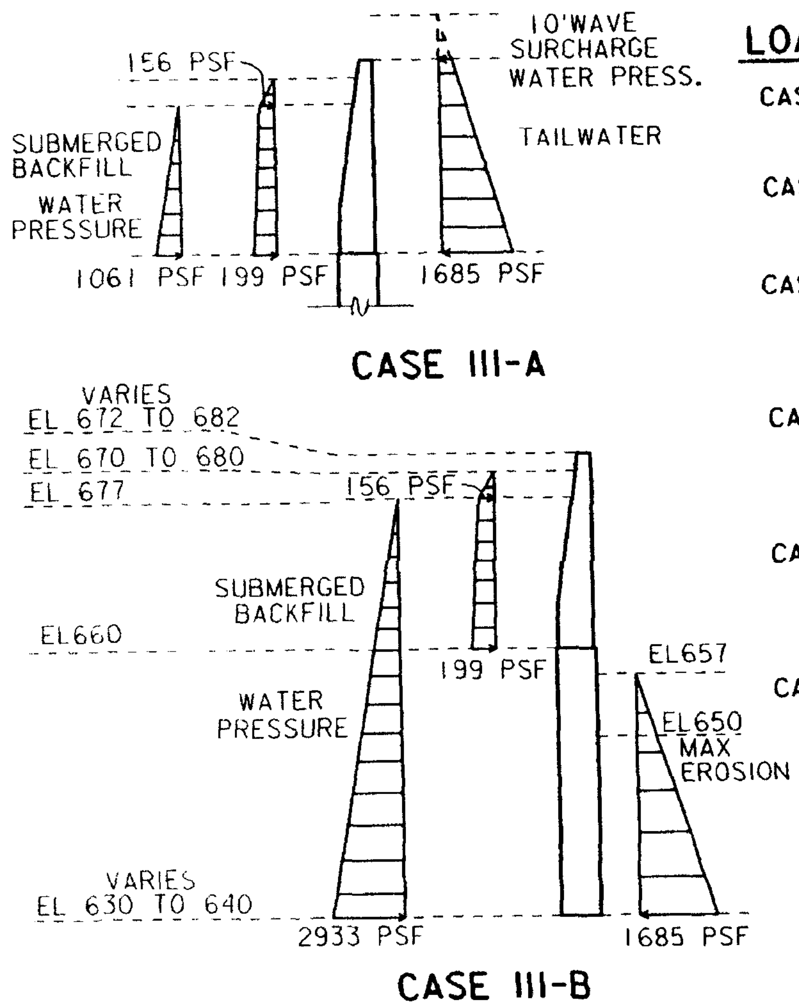


Figure 3. I-Wall section



LOAD CASE LIST

CASE I-A CONSTRUCTION CONDITION, WIND LOAD ONLY
1. 30 PSF WIND LOAD
2. NO BACKFILL

CASE I-B CONSTRUCTION CONDITION, TAILWATER FLOODING
1. TAILWATER EL.677
2. NO BACKFILL

CASE II-A NORMAL OPERATING CONDITION, MAX. EROSION
1. 30 PSF WIND LOAD
2. MOIST BACKFILL TO EL.680 MAX., 670 MIN.
3. TAILWATER EL.650 (LOW OR DEWATERED)

CASE II-B NORMAL OPERATION CONDITION, NO EROSION
1. 30 PSF WIND LOAD
2. MOIST BACKFILL TO EL.680 MAX., 670 MIN.
3. LANDSIDE WATER EL.670, TAILWATER EL. 660

CASE III-A SPILLWAY FLOOD CONDITION, WAVE SURCHARGE
1. MOIST BACKFILL TO EL.680 MAX., 670 MIN.
2. WATER TO EL.677 ON LANDSIDE
3. TAILWATER EL.677 PLUS 10' SURCHARGE

CASE III-B SPILLWAY FLOOD CONDITION, WAVE SURCHARGE
1. MOIST BACKFILL TO EL.680 MAX., 670 MIN.
2. WATER TO EL.677 ON LANDSIDE
3. TAILWATER EL.677, MINUS 20' SURCHARGE

CASE III (CRITICAL CASE)

SPILLWAY FLOOD CONDITION

Figure 4. Load cases

structurally adequate. Contractors and equipment suppliers and experts were now consulted to ascertain the feasibility of each method.

Method 1, trenching, was rejected because it is not possible to trench rock to an adequate depth (i.e., approximately 30 ft from the top of rock at el 660.0 to el 630.0). Maximum trench depth is 14 ft with available equipment.

Method 2 resulted in heavily reinforced piers which were drilled directly adjacent to each other. Drilling tolerance over a 30-ft depth could not ensure that the drill bit would not encroach on a previously drilled and filled pier to the point of contacting reinforcement. Contact with reinforcement would destroy a very expensive drill bit. Also, the availability of a 40-in.-diam bit in a timely manner was questionable. Therefore, Method 2 was rejected.

Method 3 appeared to meet the requirements. The drill bit was readily available, the piers would be staggered and separated adequately for drilling tolerances, and the required 30-ft depth was easily attainable. Method 3 was adopted.

Analysis and Design of I-Wall

The analysis and design of the I-wall presented some unique challenges. Three distinct analysis methods were used to attempt to model the anticipated behavior of this wall due to various load cases. Several Corps of Engineers computer programs and one non-Corps program were used for analysis and design. See REFERENCES for a complete list of these programs and a list of criteria used for loading and design.

Analysis methods (Figure 5)

The first method, done by hand, was the simplest, most conservative approach. The moments, axial forces, and shears were calculated based on the controlling load cases. The maximum moment and shear at el 650.0 were adjusted for a length of wall equivalent to one pier and applied directly to one pier section. This method considers the piers to be fixed at the rock surface and discounts any frame action due to the piers being staggered.

The second method, utilizing and comparing the Corps sheet-pile computer program CSHTSSI and the LPILE program, models the entire height of the pier as a pile embedded in rock with properties closely approximating those at the construction site. The upper wall is modeled with its actual section properties, and the water and soil loads are placed directly on the model. This method distributes the moment over the length of the pier with the maximum moment located near the rock surface and an inflection point and reversed moment farther down the pier. This method also discounts any frame action.

The third method, utilizing the Corps CFRAME computer program, fixes the piers at el 650.0, but considers the frame action due to the staggering of the piers. The result is that the moment at the base of the piers is resisted in part by a tension/compression couple. This method assumes fixity of the piers to the upper wall.

Results of analysis methods

The results of the three analysis methods correlated well and were as expected. The results of Analysis Method 1 were used to determine the controlling load cases (Load Cases III A and B). These load cases were then analyzed using Analysis Methods 2 and 3. Table 1 shows the results obtained from the three analysis methods for the controlling load case (Load Case III B).

The moments for Method 1 were the most severe, since both the frame action of the staggered piers and the length effect of the piers below the rock surface were ignored. As expected, Method 2 resulted in less severe moments than Method 1 since the length effect of the piers was considered by CSHTSSI and LPILE. Essentially, the severe moment of Method 1 was distributed over the length of the pier in Method 2. Method 3, which considered

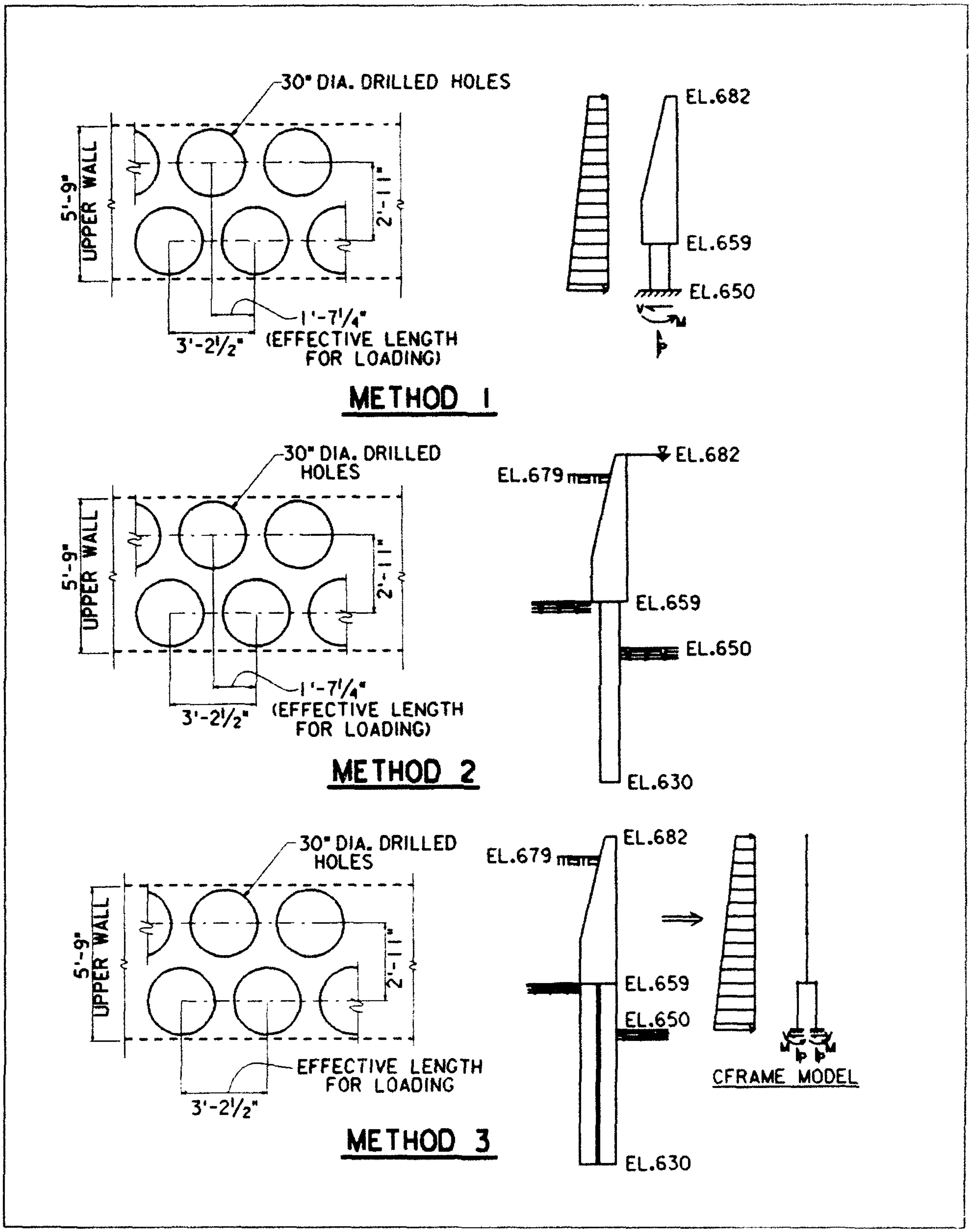


Figure 5. Analysis methods

Table 1 Analysis Results, Maximum Values[1]					
	Analysis Method 1	Analysis Method 2 LPILE	Analysis Method 2 CSHTSST	Analysis Method 3 Pinned at Pier/Wall	Analysis Method 3 Fixed at Pier/Wall
Section 1 (Base of Upper Wall)					
Shear (lbs)	12,264	16,000	14,011	33,670	39,800
Moment (lb-ft)	98,366	108,333	87,397	227,250	263,583
Section 2 (Pier at el 650)					
Shear (lbs)	28,000	240,000	52,000	118,500	188,000[2]
Moment (lb-ft)	503,583	383,750	414,200	268,750	241,167[2]

[1] Load Case III-B (backfill water level at el 677, tailwater at el 657 (wave trough)). Only results for high wall (el 682) are shown.
[2] These results were used in final design.

the frame action of the staggered piers but ignored the length effect of the piers, resulted in the least severe moments.

It is evident that the frame action was more critical to the analysis than the length effect of the piers. Therefore, the exclusion of length effect from Method 3 essentially provided an additional safety factor for this analysis method.

After considerable discussion, debate, and thought, it was decided to adopt Method 3 as the analysis method to use for final design. Because of the extreme controlling load cases, it was decided that Methods 1 and 2 were overly conservative. The reinforcement in the piers was detailed as extending a full 5 ft-0 in. into the upper wall to ensure a fixity at this juncture and ensure that the frame action will occur.

In addition, using the CFRAME model, the erosion was extended downward to el 640.0, exposing 10 additional feet of pier and reducing the length socketed in rock to 10 ft. This configuration failed. Although the design erosion was to el 650.0, provision was made for future installation of rock anchors at el 655.0, which will anchor the channel-side piers into the landside rock. Pipe sleeves were installed through the pier reinforcement for future underwater drilling and installation if the need arises.

Finally, the shear in the channel-side rock was checked. Using an allowable shear, C , of 5.4 tons/square foot and a width of rock shear plane of 40 ft, the safety factor was calculated to exceed 8.0. The C value for sliding failure was from the Truman Dam Spillway Design Memorandum and is a low value based on the weakest anticipated rock plane.

Because of the massive size of the upper wall in order to fully encase the staggered piers and the reduced loading toward the top of the wall, temperature reinforcement requirements controlled overstress requirements. Because of the size of the upper wall, however, temperature reinforcement (based on EM 1110-2-2103, May 1971) was at maximum amounts.

Design

After analysis was completed, the moments, axial loads, and shears for critical sections of the piers and wall were tabulated. The Corps program CGFAG was used for the design. This program allows for the approximation of the round shape of the piers and round reinforcement cages. The design for the piers was checked with the CRSI Manual tables for circular columns. The results correlated.

Construction of I-Wall

The clearing of the construction area began in mid-October 1990, and the completion date of the contract is scheduled for 15 July 1991. The contract was awarded to the Osage Bridge Company of Fulton, MO. Osage Bridge let

subcontracts to *Sheffield Steel Company* to fabricate the reinforcement, and Midwest Foundations of Topeka, KS, to do the drilling. Howard Redi-Mix of Warsaw, MO, supplied the concrete. The notice to proceed was given on 19 September 1990.

Piers (Figures 1, 2, and 3)

The pier drilling and concrete placement commenced on 11 December 1990 and were completed on 12 April 1991. Eighty-three 30-in.-diam holes, 29 ft deep, were drilled, and four 30-in.-diam holes, 20 ft deep, were drilled. A total of 423 cu yd of concrete and 65 tons of reinforcement were placed into the pier holes. The contractor surveyed hole locations and fabricated a structural steel framework and moveable templates to act as drill guides. It took 2 to 3 hr to drill one hole. The contractor drilled two to three holes per day. The cuttings came up as a slurry and were suctioned off into a truck for disposal. After a hole was drilled, a temporary steel pipe casement was placed into the hole until reinforcement and concrete placement. Drilling could not begin on a hole directly adjacent to a previously drilled hole until the first hole was filled with concrete and the concrete had cured to a tested strength of 3,000 psi. Also, drilling could not begin closer than 20 ft to any freshly placed concrete until the concrete reached a strength of 3,000 psi. These restrictions caused the contractor to carefully coordinate his drilling and concreting operations.

The concrete specified for the piers was high early strength using Type III cement and a slump of 6 in. The concrete was pumped into the holes since 10 to 20 ft of standing water was present in the holes after drilling. The concrete was pumped to the bottom of the hole through a pipe which was withdrawn as the concrete level rose. The outlet end of the pipe was kept embedded approximately 10 ft below the surface of the concrete. The water was displaced out of the hole by the concrete and pumped away from the shoreline.

Upper wall (Figures 1, 2, and 3)

Construction of the upper portion of the wall began on 13 April 1991 and was completed in June 1991. This upper portion of the wall above the piers was placed conventionally. The quantity of concrete for the entire upper wall was 820 cu yd with 22.5 tons of reinforcement. Standard concrete with a 2-1/2-in. slump was placed into the forms from the top using concrete buckets and tremie pipes. The contractor chose to use steel formwork which was stripped and reused for successive wall sections. The forming for the Transition Section of wall proved to be the most difficult since the height of wall and the slope of the landside face varies. The contractor chose to continue the use of steel formwork in this area and continually vary the sloped face. The angle on the stiff back exterior form braces was varied. The braces were set in place, and the steel plate form was bolted tightly to the braces, forcing the form plates to assume the "warped" shape. The vertical reinforcement also had to vary in order to maintain the proper concrete cover.

L-wall

The design of the L-wall was conventional and basic and is not discussed in this paper. The concrete placement also was conventional. However, before placement of this portion of the wall, 17 cu yd of existing concrete had to be excavated and removed from behind the existing spillway training wall. The magnitude of this removal was a change from the existing conditions shown in the plans for the dam and proved to be one of the most difficult and tedious portions of this project.

Air-powered jackhammers and a backhoe were used to accomplish the work, and repair of the landside surface of the existing wall was required before placement of the L-wall.

Equipment

The following is a list of the major equipment used by the contractor:

Drill - Ingersol-Rand Model DHD-130A Downhole Hammer Drill, with 30-in. Button Bit, Hain Model 4-71/3531-3 Crane-Mounted Drill Attachment

Crane (for drill and other uses as required) - Manitowac Model 2900WC 65-Ton Liftcrane

Vacuum Truck (used to vacuum drill cuttings and water as holes were drilled) - 2,800 cfm, 5+ cu yd capacity with 8-in.-diam line

Concrete Pump Truck - Schwing Model 1200/36 with a 28-m boom lift capacity and a 4-in.-diam pipe

Although the contract stated that the contractor would have to plan his work for high tailwater elevations due to powerhouse generation, he was fortunate to have extremely low tailwater during the duration of the construction. With the exception of the pier concreting and a small portion of the "trenched" wall, all concrete work was done in dry conditions. There were very few days, even in winter, when the weather posed a problem. Experienced, innovative contractors and good preparation turned a potentially complex and difficult construction problem into an excellent, smooth-running project. The end product is of sound design and construction and should eliminate the downstream left bank erosion problems at Truman Dam.

References

Computer Programs with Manuals

Computer-Aided Structural Engineering (CASE) Programs:

CFRAME - Computer Program with Intractive Graphics for Analysis of Plane Frame Structures (3 June 1986)

CSHTSSI - Computer Program for Soil-Structure Interaction Analysis of Sheet-Pile Retaining Walls (June 1983)

CGFAG - Computer Program, Concrete General Flexure Analysis with Graphics (30 May 1986)

Private Industry Program:

LPILE1 by Ensoft, Inc., Austin, TX

Criteria Used for Loading and Design

EM 1110-1-2101, "Working Stresses for Structural Design," November 1983.

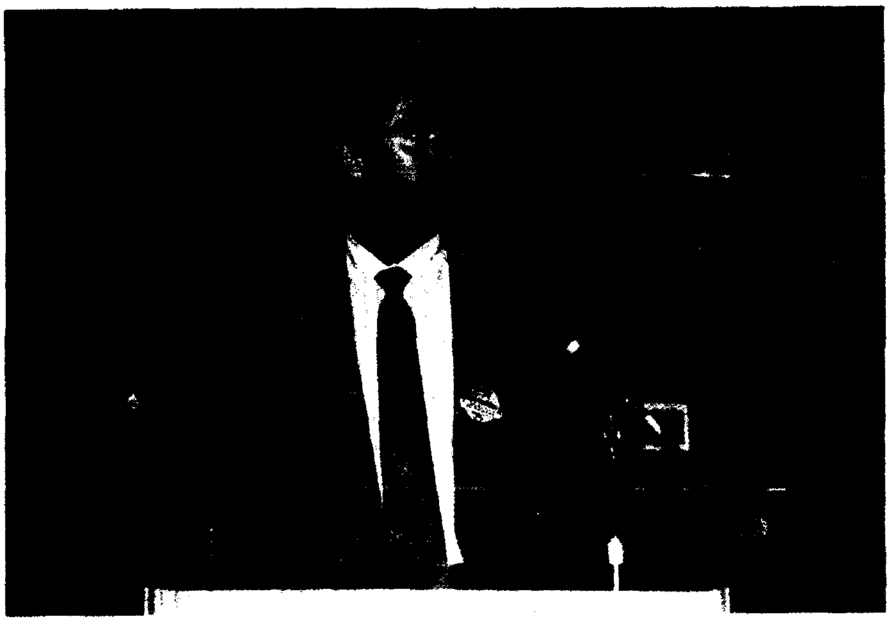
Draft EM 1110-2-2502, "Retaining and Flood Walls," September 1987.

EM 1110-2-2200, "Gravity Dam Design," September 1958.

ETL 1110-2-312, "Strength Design Criteria for Reinforced Concrete Hydraulic Structures," March 1988.

Concrete Reinforcing Steel Institute Handbook, 2nd edition, 1975.

ACI 318-89/318 R-89, "Building Code and Commentary."



The Engineer's Role in Urban Search and Rescue

by

David Hammond, PE,[1] *Edward Hecker,*[2] *Richard Young,*[3] *and Kelley Aasen, PE*[3]

Abstract

Recent disasters have illustrated the vulnerability of certain types of structures in heavily populated areas and have served to bring Urban Search and Rescue (US&R) to the forefront of response planning. The collapse of these structures during major earthquakes poses unique challenges to rescue personnel and demonstrates the need for specific engineering skills within the response organization. The Corps of Engineers has been formally tasked by Forces Command (FORSCOM) to provide engineers in support of their US&R mission under the Federal Response Plan. During US&R operations, the engineer will provide ongoing support to the search and rescue elements. Engineering advice may be required to refine search routes or evaluate shoring and stabilization systems. Additional engineering advice may be needed regarding construction materials and breaching operations. Recognizing the limited first responder experience of Corps personnel, Headquarters, US Army Corps of Engineers, directed the newly established Earthquake Preparedness Center of Expertise to develop a training program for responding personnel. This coincides with the development of specialized US&R programs by the State of California and the Federal Emergency Management Agency. This paper delineates the role of the engineer in support of US&R and how the Corps will help develop the required engineering resources for this mission.

Introduction

Recent disasters have illustrated the vulnerability of heavily populated urban areas and have served to bring Urban Search and Rescue (US&R) to the forefront of response planning. Our modern heavy structures pose unique challenges to rescue personnel and have demonstrated the need for engineering skills within the rescue organization. In recognition of this, the new Federal and State of California urban search and rescue programs list Structural Specialists as part of every US&R Task Force.

In addition, under the Federal Disaster Response Plan, Forces Command (FORSCOM) has formally tasked the Corps to provide engineers in support of their US&R operations. This tasking will require the Corps to maintain a cadre of engineers ready to accomplish this mission.

This tasking immediately invites two key questions: What is the role of the engineer in support of US&R, and how does the Corps prepare for its US&R mission?

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The Engineer's Role

US&R is a high risk undertaking, conducted in a building that is fully or partially collapsed. These buildings will normally be multi-storied, containing heavy debris with a high potential for additional collapse. This is the work environment of the US&R Task Force. Engineers trained and experienced in damaged building evaluation can help reduce the risk to these task forces and to victims. In order to function effectively, these engineers must be well prepared to make difficult value judgments in an environment that is very different from the orderly design office.

Defining the problem

As in most engineering tasks, the first job is to identify the problem. In the case of a collapsed structure, there are some fundamental questions to be addressed. Some of the answers may be available prior to conducting the initial assessment; however, most will require on-scene investigation. The answers to these questions may be vague and uncomfortable for the engineering mind to assess, but value judgments will have to be made. Often these judgments will be made under severe pressure. The essential information required is:

- **What caused the collapse?** Was it an earthquake, wind (hurricane or tornado), construction failure, explosion/fire, flood, or landslide? Each of these has special consequences and produces unique problems to specific structural types. Earthquakes affect heavy and poorly connected structures most severely. Aftershocks can produce further collapse during rescue operations. Strong wind storms normally pull lighter materials off buildings, but also cause devastation from flying objects and tidal surges. Construction failure and structural decay collapse often involve column or other vertical stability failures. Explosions have very different effects on light frame compared to heavy structures.

- **What type (or types) of structure is involved?** Each type can produce a different collapse pattern and set of post-collapse hazards, and any one building can contain a combination of types.

  * **Light Frame.** Usually of wood-frame, box-type construction up to four stories in height, used for living or other occupancy. Failure is in the skin that covers the frame. Plumb walls rack and become parallelograms.
  * **Heavy Wall Buildings.** Buildings with unreinforced masonry walls, tilt-up concrete walls, or other masonry walls. They may be up to 10 stories high and used for living, commercial, and/or industrial occupancies. Failure is in connections of wall to floors/roof or within the wall of these box-type buildings.
  * **Heavy Floor Structures.** Concrete frame buildings up to 20 stories and highway structures. They may have a few concrete walls. Failure is most often in columns at joints with floors. Many failure types have been observed (pancake, story offset, weak story/soft story, torsion failure of corner building, and even overturned buildings.
  * **Precast Buildings.** Buildings assembled of factory built, mostly lightweight concrete parts. They range from 1 to 20 stories and are mostly frame type, but may have some concrete walls. Failures are usually in interconnection of parts, which can result in partial or total collapse.

- **What types of hazards are present?** Generally, the Structural Specialist is concerned with falling hazards (parts of building or debris that may fall), collapse hazards (existing cavity may become smaller), and other hazards (gas, toxics, water, etc.). Each structural type will present a somewhat different set of hazards. The engineer must use his knowledge of materials and attempt to identify the possibility of brittle versus ductile behavior. As an example, the hazard presented by a leaning plaster-covered

unreinforced masonry partition is quite different from one of wood/metal studs and plaster.

- **What are the location and condition of remaining voids in the structure?** Are they likely to collapse in an aftershock? Is it shorable? Can it be entered from the top? If there is a basement, what is its condition?

- **What are the locations of all previous access holes in the structure?** To the extent possible, vertical and horizontal shafts should be identified, including: entries, stairs, elevator shafts, duct openings, and vertical pipe openings.

- **What tools and shoring materials are available?** The engineer must determine if it is feasible to shore the structure and if the structure can be effectively cut.

- **What are the search and rescue capabilities of the team?** Can remote search methods be used to prioritize the site? Should the team refer the site to others?

Application

The information gathered during the initial assessment process will be presented to the Task Force Leader. The engineer's recommendations for stabilizing the structure will be essential to the overall safety of search and rescue personnel. Locations of potential voids and ingress routes will serve a vital role in determining the initial search strategy.

During operations, the engineer will provide ongoing support to the search and rescue elements. Engineering advice may be required to refine search routes or evaluate shoring and stabilization systems. Additional engineering advice may be needed regarding construction materials and breaching operations.

The US&R incident will require continued reassessment due to aftershocks, load readjustment due to shoring, and reprioritization of site due to search results.

The duties of the Structural Specialist are complex and challenging. What are the prerequisites for an engineer to participate in this program?

What it takes

Although in many situations a Structural Engineer may be preferable, those engineers with US&R experience agree that Civil and General Engineers can be trained to support US&R. The key in evaluating Structural Specialist candidates is their familiarity with building construction and experience in structural design and analysis. This experience should include evaluation of existing structures, field investigation, or construction observation. The candidate should possess general knowledge of the design and construction techniques for wood, masonry, concrete, and steel with some knowledge of their behavior under adverse loading conditions.

Given the urban search and rescue environment, the Structural Specialist must be physically fit and capable of improvising and functioning under extremely adverse field conditions. Structural Specialists must be self sufficient for 72 hr and able to function for long hours, on or around rubble, at heights, and in confined spaces.

The Structural Specialist must understand safe working practices and procedures as required in the urban disaster environment. Current certification in Advanced First Aid and Cardiopulmonary Resuscitation is essential. The Structural Specialist should be familiar with critical incident stress syndrome management strategies and hazardous materials awareness. Structural Specialist will be required to be available on short notice and able to mobilize within 6 hr of notification.

This description provides insight into the nature of Corps missions in response to the FORSCOM tasking.

The Corps Mission

Under the Federal Response Plan, FORSCOM serves as the Executive Agent for the Department of Defense regarding their support for domestic natural disasters. In this capacity, FORSCOM serves as the lead Federal agency for Urban Search and Rescue (ESF #9). Within the Federal Response Plan, the Corps serves as the lead Federal agency for Public Works and Engineering (ESF #3). It is within this framework that FORSCOM has tasked the Corps to provide engineers for support of their US&R operations.

Current FORSCOM plans call for the Corps to provide blocks of engineers in 12-hr increments over a 72-hr period. In order to meet this requirement, the Corps must maintain a cadre of 100 to 200 engineers ready to support US&R.

The training program

Headquarters, US Army Corps of Engineers, recognized that specific engineer/rescue training was not readily available within the United States and that such training would have to be developed. In January 1991, the Corps' South Pacific Division submitted a draft engineer/US&R training proposal for Headquarter's review. Headquarters directed that the newly established Earthquake Preparedness Center of Expertise (EQPCE) develop the training program to meet the requirement (Table 1).

This coincided with the development of specialized US&R programs by the State of California and Federal Emergency Management Agency (FEMA). The integration of engineers into the US&R process was a goal of each program, with each agency recognizing the necessity of specific engineer/US&R training.

Although US&R operations undertaken by FORSCOM organizations will be less complex than those addressed by the specialized State and Federal programs, the engineering skills are similar. The technical engineering data collected for this project will serve as a solid foundation for the more complex requirements of the specialized programs. Using this curriculum as a base, the EQPCE is prepared to develop the more complex engineer training as a follow-on to this project. This course would then become a prerequisite for the more specialized engineering course. Such an approach allows for greater continuity between the curriculum, a more cohesive overall program, and greater cost effectiveness. Recognizing the limited "first responder" experience of Corps personnel, this program will be as comprehensive as possible. The goal is to provide the engineer with sufficient response skills to facilitate their integration into the US&R organization. Table 1 shows the tentative training schedule and provides a brief description of the courses.

Development

This program will be developed in conjunction with criteria established by FEMA and the State of California (Office of Emergency Services) and will be consistent with the criteria established by their respective programs. This continuity will be enhanced by the appointment of a liaison between the Corps and California OES. A detailed plan of action has been developed and is being coordinated with these agencies to assure that the broadest possible applicability and utility is achieved as the training program proceeds to implementation.

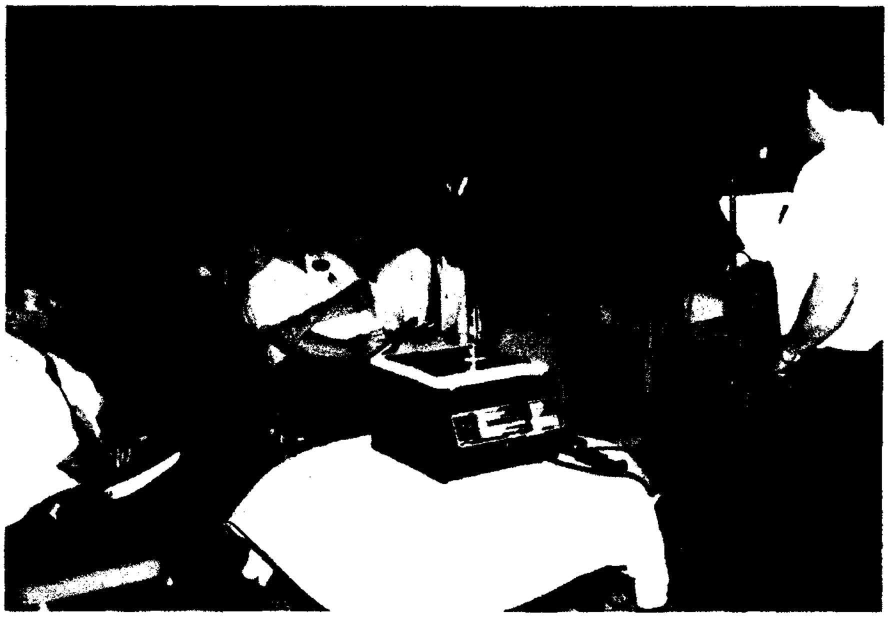
Recruitment

A survey/interest form is being provided to attendees to help develop an initial inventory of US Army Corps of Engineers personnel who may want to participate in this important program. It is also intended to select and train several Structural Specialists to serve as Regional Team Leaders who can, in turn, help expand the training base. An individual who is selected and trained for this mission must be available for immediate deployment to a disaster site should FORSCOM implement its response plan. Individuals who respond to the survey are not obligated to enter this program and will receive further information as the program develops. A formal recruitment process will be initialed late in FY 91.

Table 1
Engineeer Training, US&R, Draft Schedule[1]

Week One	
Sunday	Travel & Check-in
Monday	Welcome
	Course Overview
	Introduction to US&R
	First Aid
Tuesday	Rescue Systems I
Wednesday	Rescue Systems I
Thursday	Rescue Systems I
Friday	Rescue Systems I
Saturday	Off
Week Two	
Sunday	Off
Monday	Rescue Systems I
Tuesday	ATC-20
	Structural Collapse
Wednesday	HAZMAT First Responder
Thursday	Advanced Shoring
	Critical Stress Management
	Review
Friday	Field Exercise
	Debriefing/Evaluation
	Check-out & Travel to HOR

[1] Training Day begins at 0800 and ends at 1700.



The Corps of Engineers and ATC-20

by

Jim Tanouye, PE,[1] *and Jim Couey, PE*[2]

Abstract

In the aftermath of an earthquake occurring in a heavily populated region, there would be a need for damage inspectors to perform safety evaluations of standing buildings. Experience has shown that local building departments become quickly overloaded by the volume of safety evaluations required and must seek additional personnel. Although many of the individuals pressed into service are associated with building design and construction trades, many are not structural engineers. ATC-20 was developed by the Applied Technology Council for two purposes: to provide uniform guidelines and procedures to perform safety evaluations of common building types; and to provide a basis of training for damage inspectors who are not structural engineers. With a large engineering staff available as a resource, the US Army Corps of Engineers (Corps) could provide personnel for training and subsequent deployment as damage inspectors upon request by local and state governments. In addition, the Corps could consider the adaptation/adoption of ATC-20 for use on military installations.

Introduction

In the immediate aftermath of an earthquake disaster that could occur in a heavily populated region, there would be a need for damage inspectors to perform safety evaluations of the standing buildings. Safe buildings need to be identified for their use as shelters or for the continuation of their normal functions. Unsafe buildings need to be identified to prevent their use and the resultant injuries or loss of life that might occur due to hazardous conditions created by the earthquake. The primary responsibility to perform these safety evaluations lies with the local building departments. Experience from recent earthquakes has shown that local building departments become almost immediately overloaded with the volume of building safety evaluations required and must seek additional damage inspectors to alleviate the backlog. An undesirable alternative would be to extend the time it would take to perform all the needed building safety evaluations, but this would risk incurring injuries or loss of life from the use of any unsafe buildings.

Background

In the aftermath of an earthquake disaster that has occurred within their jurisdiction, local building departments would normally notify their respective state governments of the need for additional damage inspectors. In turn, the state government would then coordinate and consolidate these needs and issue a public request for individuals to serve as damage

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inspectors. Individuals would respond from a variety of sources within the region the earthquake disaster occurred; namely, employees of local, state, and Federal governmental agencies, such as the Corps of Engineers (Corps), as well as numerous private agencies, including construction firms, design firms, and professional affiliations, such as the American Society of Civil Engineers. Many individuals from similar sources outside the region of the earthquake disaster would respond as well.

Although many of the individuals who volunteer for service as damage inspectors or who are pressed into service as damage inspectors are associated with the various building design and construction trades, many of them are not structural engineers by practice or training. In addition, many of the governmental and private agencies from which these individuals originated have their own unique procedures and guidelines for determining whether or not a particular building is safe, as well as their own unique categories for posting (i.e., safe, limited entry, emergency use only, unsafe) a building as a result of the safety evaluation. The lack of a structural engineering background in conjunction with the origination of the individual has led to two observations regarding the use of these individuals as damage inspectors: the lack of uniform procedures for damage inspectors to assist them in determining whether or not a building is safe; and the lack of consistency by different damage inspectors in concluding whether or not a particular building is safe and in what category to post that building.

As a result of these two observations, the Disaster Emergency Services Committee of the Structural Engineers Association of Northern California initially proposed a project to: develop and document qualitative procedures and guidelines for the safety evaluation of standing buildings damaged by an earthquake; and to develop and document appropriate training materials describing these procedures and guidelines. The California Governor's Office of Emergency Services, California Office of Statewide Health Planning and Development, and Federal Emergency Management Agency jointly awarded a contract to the Applied Technology Council (ATC) in July 1987 to develop procedures for the postearthquake safety evaluation of buildings. The Applied Technology Council in turn developed and subsequently published *ATC-20, Procedures for Postearthquake Safety Evaluation of Buildings* (ATC 1989a) and *ATC-20-1, Field Manual: Postearthquake Safety Evaluation of Buildings* (ATC 1989b). The purpose of ATC-20 and ATC-20-1 is to provide local building departments the procedures and guidelines by which to perform safety evaluations of common types of buildings encountered in the United States. These procedures and guidelines are intended to promote consistency in the rating of damage of a particular building so that different damage inspectors will arrive at the same conclusion regarding its level of safety and in what category to post that building.

It should be noted the ATC is a nonprofit, tax exempt corporation established in 1971 by the Structural Engineers of California for the purpose of assisting the practicing structural engineer in the task of keeping abreast of and utilizing technological developments in the field of structural engineering.

Evaluation System

In the development of ATC-20 and ATC-20-1, consideration was given to the need to conserve and judiciously use the services of the limited number of structural engineers that would be available in the aftermath of an earthquake disaster. A three-level system was developed to perform safety evaluations of buildings (Figure 1). The first-level procedure is the Rapid Evaluation. The goal of the Rapid Evaluation is to quickly designate through visual inspection the apparently safe and obviously unsafe buildings. Those buildings not designated as safe or unsafe, the questionable buildings, go on to the next level. The second-level procedure is the Detailed Evaluation. The goal of the Detailed Evaluation is to designate through visual inspection the questionable buildings from the Rapid Evaluation as safe or unsafe. Those buildings

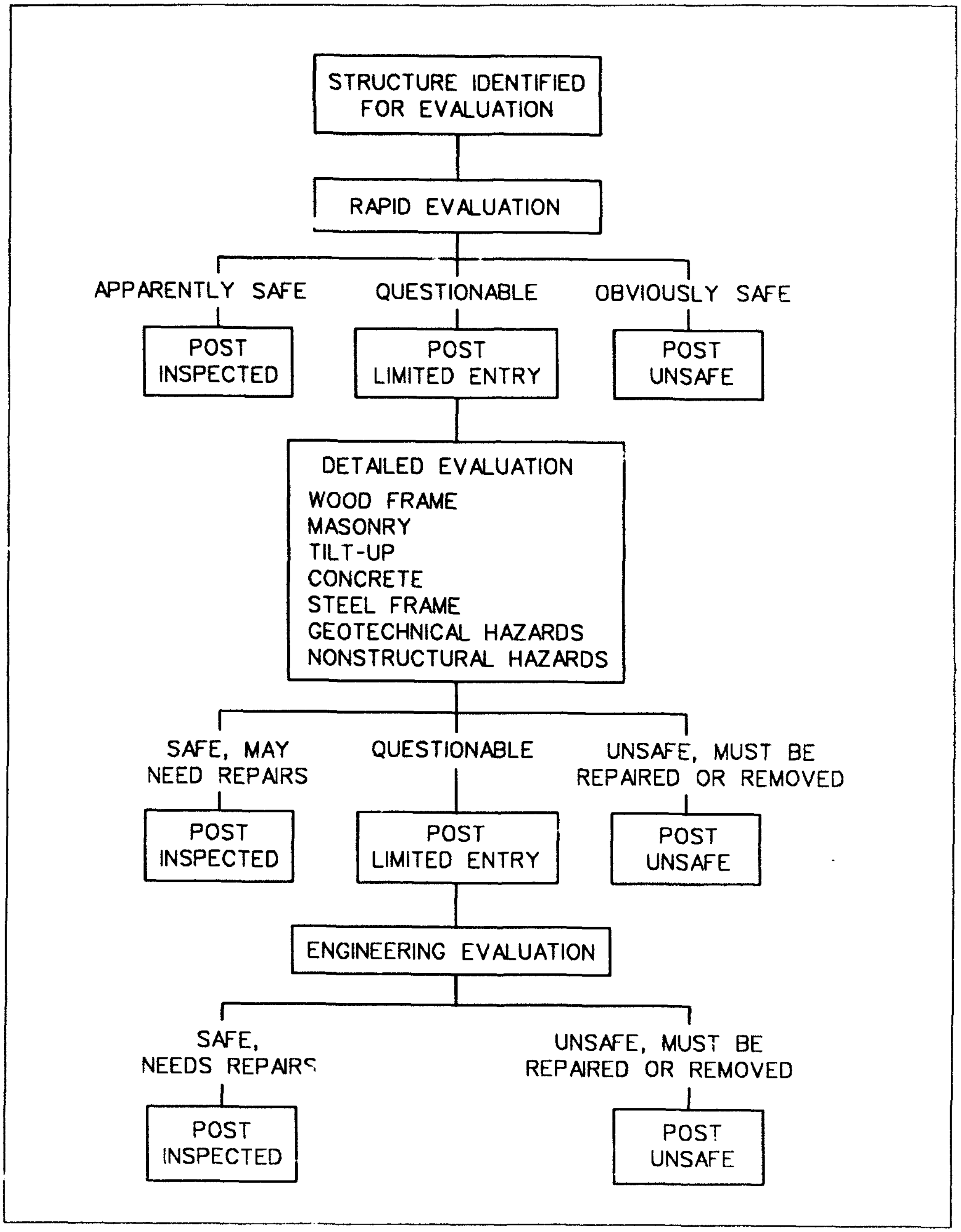


Figure 1. Three-level system of building safety evaluations

still questionable as to their safety after a Detailed Evaluation go on to the next level. The third-level procedure is the Engineering Evaluation. The goal of the Engineering Evaluation is as its title implies; to perform an in-depth engineering evaluation of the questionable buildings from the Detailed Evaluation where safety can not be determined by visual inspection alone.

Posting

The posting categories developed by ATC-20 and ATC-20-1 correspond to the safety evaluation of the buildings. An apparently safe building is posted INSPECTED, a questionable building is posted LIMITED ENTRY, and an unsafe building is posted UNSAFE (Table 1).

Table 1
Posting Categories

Posting Classification	Color	Description
INSPECTED	Green	No apparent hazardous condition present. No restriction on use or occupancy.
LIMITED ENTRY	Yellow	Hazardous condition believed to be present. Use for emergency purposes only; no occupancy on continuous basis. Possible aftershock hazard.
UNSAFE	Red	Hazardous condition present. No use or occupancy allowed. Imminent collapse from aftershock.

Rapid Evaluation

The goal of Rapid Evaluation is to quickly, and with a minimum staff, visually inspect and evaluate buildings within the damaged area. Rapid Evaluation was developed to utilize the services of individuals with at least 5 years of general experience in the building design and construction trades, thus conserving the services of structural engineers for performance of Detailed Evaluations. Rapid Evaluation was developed to be performed within a time span of 10 to 20 minutes per building.

Rapid Evaluation consists of the following procedures:

- Examine the entire outside of the building.
- Examine the ground in the general area of the building for fissures, bulges, or slope movement.
- When the building can not be viewed sufficiently from the outside, enter the building for a cursory view of the inside. Do not enter obviously unsafe buildings.
- Complete the Rapid Evaluation form using the six Rapid Evaluation criteria (Appendix A and Table 2, respectively).
- Post the building according to the results of the evaluation.
- Explain the significance of the LIMITED ENTRY or UNSAFE postings to the building occupants, and advise them to leave the building immediately.

Table 2
Rapid Evaluation Criteria

Condition	Posting
Building has collapsed, partially collapsed, or moved off its foundation.	UNSAFE
Building or any story is significantly out of plumb.	UNSAFE
Obvious severe damage to primary structural members, severe racking of walls, or other signs of severe distress present.	UNSAFE
Obvious parapet, chimney, or other falling hazard present.	UNSAFE
Large fissures in ground, massive ground movement, or slope displacement present.	UNSAFE
Other hazards present (i.e., toxic spill, fallen power line, asbestos contamination, broken gas line).	UNSAFE

Detailed Evaluation

The goal of Detailed Evaluation is to visually inspect and evaluate the buildings posted LIMITED ENTRY by Rapid Evaluation.

Detailed Evaluation was developed to utilize the services of structural engineers with at least 5 years of building design. Detailed Evaluation was developed to be performed within a time span of 1 to 4 hours per building.

Detailed Evaluation consists of the following procedures:

- Survey the building from the outside.
- Examine the building site for geotechnical hazards.
- Inspect the structural system from inside the building.
- *Inspect for nonstructural hazards.*
- Inspect for other hazards.
- Complete the Detailed Evaluation form using the eight Detailed Evaluation criteria and post the building (Appendix B and Table 3, respectively).

As part of the Detailed Evaluation, there are sections of ATC-20 and ATC-20-1 that describe in detail special areas of concern for common types of buildings encountered in the United States. The types of buildings described are summarized as follows:

- Wood Frame:
  - * Residential.
  - * Commercial, institutional, and industrial.
- Masonry:
  - * Unreinforced.
  - * Reinforced.
- Tilt-up Concrete.
- Concrete:
  - * Moment-resisting frame.
  - * Shear wall.
  - * Infill masonry frame.
- Steel Frame:
  - * Braced frame.
  - * Moment-resisting frame.
  - * Prefabricated metal buildings.
  - * Frame with unreinforced masonry infill.
  - * Frame with concrete cast-in-place or reinforced masonry walls.

Table 3
Detailed Evaluation Criteria

Condition	Posting
Overall Damage:	
Collapse or partial collapse.	UNSAFE
Building or individual story noticeably leaning.	UNSAFE
Fractured foundations.	UNSAFE
Vertical Load System:	
Columns noticeably out of plumb.	UNSAFE
Buckled or failed columns.	UNSAFE
Roof or floor framing separation from walls or other vertical supports.	UNSAFE
Bearing wall, pilaster, or corbel cracking which jeopardizes vertical support.	UNSAFE
Other failure or incipient failure of significant vertical load carrying element or connection.	UNSAFE
Lateral Load:	
Broken, leaning, or seriously degraded moment frames.	UNSAFE
Severely cracked shear walls.	UNSAFE
Broken or buckled vertical braces.	UNSAFE
Other failure or incipient failure of significant lateral load carrying element or connection.	UNSAFE
P Delta Effects:	
Multistory frame building with residual drift.	UNSAFE
Degradation of the Structural System:	
Seriously degraded structural system.	UNSAFE
Falling Hazards:	
Falling hazards present.	UNSAFE
Slope or Foundation Distress:	
Base of building pulled apart or differentially settled, with fractured foundations, walls, floors, or roof	UNSAFE
Building in zone of faulting or suspected major slope movement.	UNSAFE
Building in danger of being impacted by sliding or falling landslide debris from upslope.	UNSAFE
Other Hazards:	
Spill of unknown or suspected dangerous material	UNSAFE
Other hazards (i.e., downed power line)	UNSAFE

There are also sections that describe in detail special areas of concern for geotechnical hazards, nonstructural hazards, and essential facilities.

Engineering Evaluation

The goal of Engineering Evaluation is to inspect and evaluate the buildings posted LIMITED ENTRY by Detailed Evaluation. Whether or not to conduct an Engineering Evaluation will be the decision of the owner of the building as this level of evaluation requires the services of a structural engineering consultant. Specific procedures for evaluation were not detailed in ATC-20 and ATC-20-1 since these procedures will be selected based upon the engineering judgement of the consultant.

Corps of Engineers—Source of Evaluation Personnel

As one of the largest governmental agencies in terms of engineering resources, the Corps could provide personnel for deployment as damage inspectors to local and state governments in the case of an emergency. The majority of engineering personnel provided by the Corps would be used for Rapid Evaluation, and those with a structural engineering background could also be used for Detailed Evaluation. This would entail having Corps personnel trained in the use of ATC-20 and ATC-20-1, with a database developed to track those individuals with the appropriate training for call should the need arise.

The training basically consists of a 1-day course sponsored by the Applied Technology Council which outlines the contents of ATC-20 and ATC-20-1 in the morning and evaluates an example building in the afternoon. So far, training has been conducted in the states of California and Utah. Other states where the potential for an earthquake disaster exists have shown interest in the adoption of ATC-20 and ATC-20-1.

Perhaps the Corps should consider the adoption of ATC-20 and ATC-20-1 as a means to quickly evaluate and post standing buildings on military installations in the aftermath of a earthquake disaster.

References

Applied Technology Council. 1989a. *ATC-20, Procedures for Postearthquake Safety Evaluation of Buildings.*

Applied Technology Council. 1989b. *ATC-20-1, Field Manual: Postearthquake Safety Evaluation of Buildings.*

Appendix A

Block__________ Parcel No.__________

ATC-20 Rapid Evaluation Safety Assessment Form

BUILDING DESCRIPTION:

Name: ______________________________

Address: ______________________________

No. of stories: __________

Basement: Yes ☐ No ☐ Unknown ☐

Primary Occupancy: Dwelling ☐
Other Residential ☐ Commercial ☐ Office ☐
Industrial ☐ Public Assembly ☐ School ☐
Government ☐ Emer. Serv. ☐ Historic ☐
Other______________________________

OVERALL RATING: *(Check One)*

INSPECTED (Green) ☐
___ Exterior only
___ Exterior and Interior
LIMITED ENTRY (Yellow) ☐
UNSAFE (Red) ☐

INSPECTOR:
Inspector ID____________________
Affiliation ____________________

INSPECTION DATE:
Mo/day/year____________________
Time ____________________ am pm

Instructions: Review structure for the conditions listed below. A "yes" answer to 1, 2, 3, or 5 is grounds for posting entire structure UNSAFE. If more review is needed, post LIMITED ENTRY. A "yes" answer to 4 requires posting AREA UNSAFE and/or barricading around the hazard. Hazards such as a toxic spill or an asbestos release are covered by 6 and are to be posted and/or barricaded to indicate AREA UNSAFE.

Condition	*Yes*	*No*	*More Review Needed*
1. Collapse, partial collapse, or building off foundation	☐	☐	☐
2. Building or story noticeably leaning	☐	☐	☐
3. Severe racking of walls, obvious severe damage and distress	☐	☐	☐
4. Chimney, parapet or other falling hazard	☐	☐	☐
5. Severe ground or slope movement present	☐	☐	☐
6. Other hazard present	☐	☐	☐

Recommendations:

☐ No further action required
☐ Detailed Evaluation required (circle one) Structural Geotechnical Other__________
☐ Barricades needed in the following areas: ____________________
☐ Other: ____________________

Posted at this Assessment: ☐ Yes ☐ No

Comments: ______________________________

ATC-20

Appendix B

Block__________ Parcel No.__________

ATC-20 Detailed Evaluation Safety Assessment Form

BUILDING DESCRIPTION:

Name: ____________________________

____

Address: ____________________________

____

No. of Stories: ________

Basement: Yes ☐ No ☐ Unknown ☐

Approximate Age: __________Years

Approximate Area:__________Square feet

Structural System:

Wood Frame ☐ Unreinforced Masonry ☐

Reinforced Masonry ☐ Tilt-up ☐

Concrete Frame ☐ Concrete Shear Wall ☐

Steel Frame ☐ Other ________________

____

Primary Occupancy:

Dwelling ☐ Other Residential ☐ Commercial ☐

Office ☐ Industrial ☐ Public Assembly ☐

School ☐ Government ☐ Emer. Serv. ☐

Historic ☐ Other ________________

OVERALL RATING: ***(Check One)***

INSPECTED (Green) ☐

LIMITED ENTRY (Yellow) ☐

UNSAFE (Red) ☐

INSPECTOR:

Inspector ID ________________

Affiliation________________

INSPECTION DATE:

Mo/day/year ________________

Time ________________ am pm

Instructions: Complete building evaluation and checklist on next page and then summarize results below.

Posting:	*Existing*	*Recommended*
None	☐	
Inspected *(Green)*	☐	☐
Limited Entry *(Yellow)*	☐	☐
Unsafe *(Red)*	☐	☐

Posted at this Assessment: ☐ Yes ☐ No

Existing posting by:

____

Recommendations:

☐ No further action required

☐ Engineering Evaluation required (circle one) Structural Geotechnical Other________

☐ Barricades needed in the following areas: ________________________

____

☐ Other *(falling hazard removal, shoring/bracing required, etc.)*: ________________

____

Comments *(Why posted Unsafe, etc.)*: ________________________

____

____

____

____

Sheet ____ *of* ____

ATC-20

ATC-20 Detailed Evaluation Safety Assessment Form (Continued)

Instructions: Examine the building to determine if any hazardous conditions exist. A "yes" answer in categories 1, 2, or 4 is grounds for posting building UNSAFE. If condition is suspected to be unsafe and more review is needed, check appropriate Unknown box(es) and post LIMITED ENTRY. A "yes" answer in category 3 requires posting and/or barricading to indicate AREA UNSAFE. Explain "Yes", "Unknown" findings and extent of damage under "Comments."

Condition	Hazardous Condition Exists			
	Yes	*No*	*Unknown*	*Comments*
1. *Structure Hazardous Overall*				
Collapse/partial collapse	☐	☐	☐	____
Building or story leaning	☐	☐	☐	____
Other____	☐	☐	☐	____
____	☐	☐	☐	____
2. *Hazardous Structural Elements*				
Foundations	☐	☐	☐	____
Roof/floors (vertical loads)	☐	☐	☐	____
Columns/pilasters/corbels	☐	☐	☐	____
Diaphragms/horizontal bracing	☐	☐	☐	____
Walls/vertical bracing	☐	☐	☐	____
Moment frames	☐	☐	☐	____
Precast connections	☐	☐	☐	____
Other____	☐	☐	☐	____
____	☐	☐	☐	____
3. *Nonstructural Hazards*				
Parapets/ornamentation	☐	☐	☐	____
Cladding/glazing	☐	☐	☐	____
Ceilings/light fixtures	☐	☐	☐	____
Interior walls/partitions	☐	☐	☐	____
Elevators	☐	☐	☐	____
Stairs/exits	☐	☐	☐	____
Electric/gas	☐	☐	☐	____
Other____	☐	☐	☐	____
____	☐	☐	☐	____
4. *Geotechnical Hazards*				
Slope failure/debris	☐	☐	☐	____
Ground movement, fissures	☐	☐	☐	____
Other____	☐	☐	☐	____
____	☐	☐	☐	____

SKETCH:

Sheet ____ *of* ____

ATC-20

Friday—12 July 1991

CESEC 91 — Challenge Workshop

Quality Facility Data: Cradle to Grave – Ronald L. Hollrah, Black & Veatch

CESEC91

CHALLENGE WORKSHOP

CESEC91
CHALLENGE WORKSHOP

The goals of the Challenge Workshop were to test the structural engineers with in the Corps on their emergency response techniques, to test their awareness of fatigue and fracture concepts, to emphasize steel inspection, and to emphasize the bridge safety program (page 2). Two actual problems were used - one was a Gate problem (page G-2) and the other was a Bridge problem (page B-2). Each problem had 4 independent teams working on a solution (4 gate teams and 4 bridge teams). Each team had a facilitator to guide the team members during their problem solving session. Two university professors were used as consultants because of their involvement on the actual problems in real time. A workshop coordinator was used to plan, organize and execute the challenge workshop (see organization chart on page 3).

During the problem solving session each team was required to select a team leader and recorder. The facilitators acted as the chief of the structural section, the consultants acted as field personnel to supply information or as an outside structural consultants if a team so requested and HQUSACE acted as the district engineer if called upon for a district decision (see team operation chart on page 4).

The information given to each team for solving the problem and the results of each team solution is included in the following information:

			Gate	Bridge
a.	teams and facilitator	page	G-1	B-1
b.	problem scenario	page	G-2	B-2
c.	facilitator guide	page	G-3	B-3
d.	reference material available	page	G-5	B-4
e.	team report form	page	G-6	B-5
f.	solution	page	G-10	B-12
g.	facilitator report form	page	G-13	B-14
h.	Lead Facilitator Summary Report	page	G-17	B-18

The two lead facilitators reported the results of the challenge workshop to the conference attendees on Friday. That information is summarized on page G-17 for the Gate problem and on page B-18 for the Bridge problem.

CHALLENGE WORKSHOP

GOALS

TEST EMERGENCY RESPONSE TECHNIQUES

TEST AWARENESS OF FATIGUE AND FRACTURE CONCEPTS

EMPHASIZE STEEL INSPECTION/EVALUATION

EMPHASIZE BRIDGE SAFETY PROGRAM

2

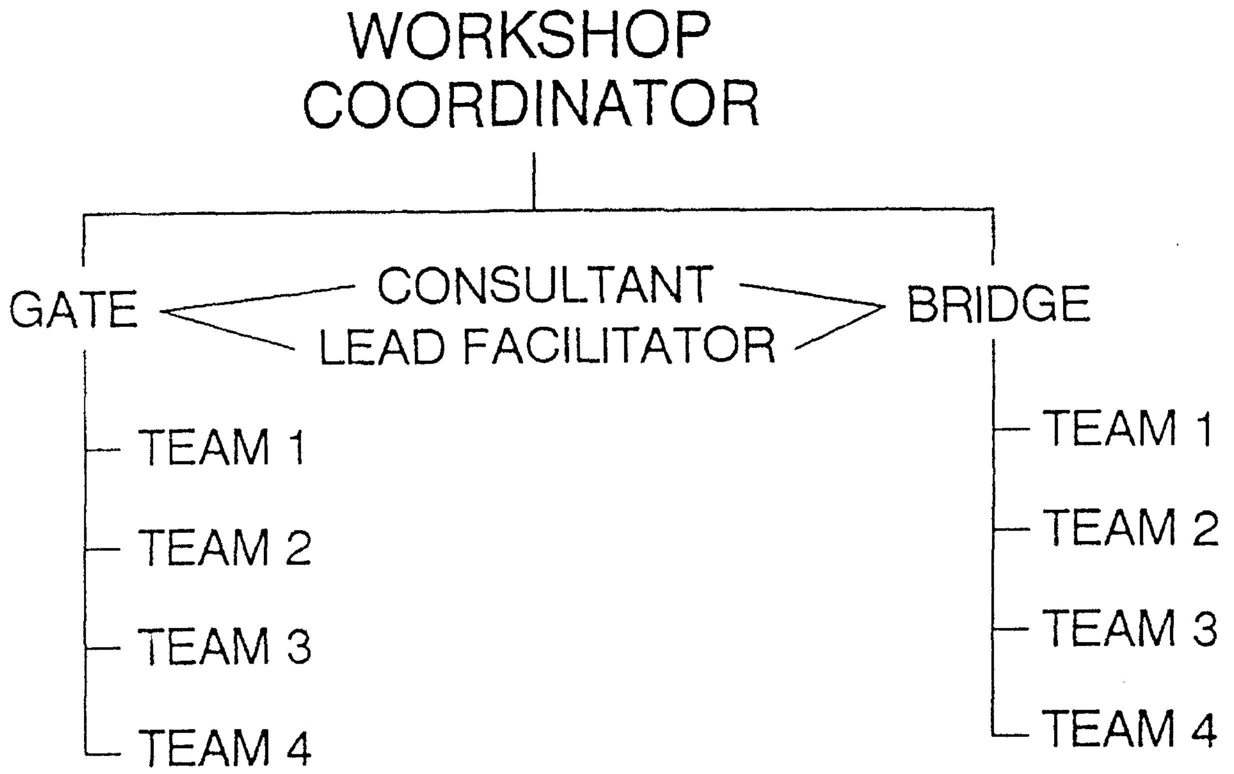
CHALLENGE WORKSHOP

ORGANIZATION

WORKSHOP COORDINATOR
Byron Foster (CESAD)

LEAD FACILITATORS
Joe Hartman (CESWD)
William Wigner (CESAJ)

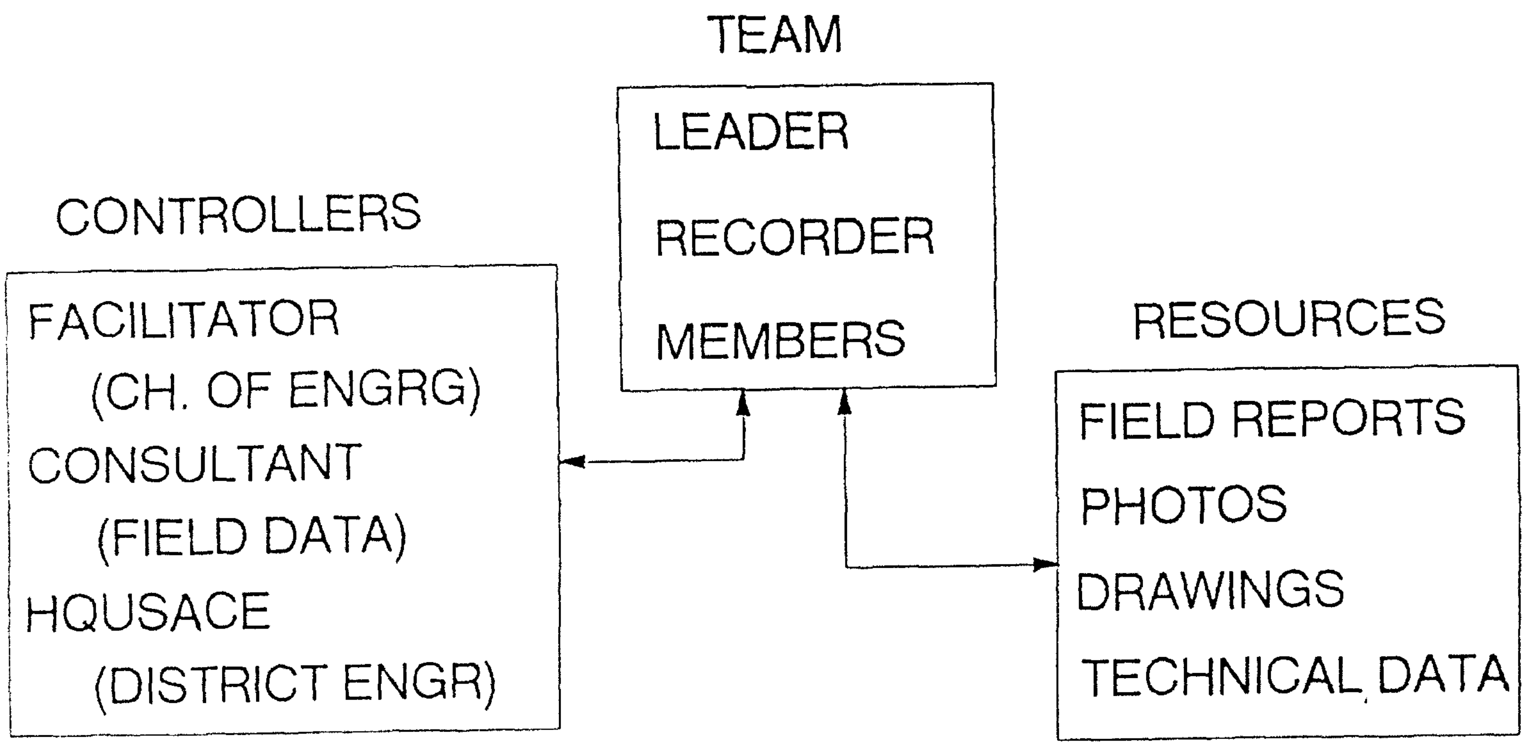
CONSULTANTS
Chon Tsai (Ohio State Univ.
Ben Yen (Lehigh Univ.)



3

CHALLENGE WORKSHOP

TEAM OPERATION



4

GATE

GATE

GATE TEAM 1	Henry Stewart - Facilitator	
	Robert E. Taylor	CEORH-ED-DS
	Eric G. Sampson	CENCC-ED-DS
	Brad Atkins	CENAO-EN-DS
	Darryl C. Bonura	CELMN-ED-DD
	William F. Strobach	CEMRK-ED-DT
GATE TEAM 2	Tom Ruf - Facilitator	
	Byron K. McClellan	CEORL-ED-D
	Frerd Joers	CENCR-ED-DS
	Paul Stroup	CESAJ-EN-DS
	Jerome Maurseth	CENPP-PE-DS
	Mark McVay	CESWT-EC-DT
GATE TEAM 3	Joe Hartman - Facilitator	
	Paul C. Noyes	CENPS-EN-DB-SD
	Leslie E. Lockett	CEORN-ED-D
	Oscar Alcoreza	CENCB-BE-SD
	Christy Hannan	CESAW-EN-DS
	Young Hsu	CELMN-ED-DT
GATE TEAM 4	Rob Kelsey - Facilitator	
	Haskell Wright	DESWL-ED-DS
	David W. Spencer	CELMS-ED-DA
	Dana Knudtson	CENPW-EN-DB-SD
	Joann McCowan	CESPL-SPL-ED-DG
	Joe Kubinski	CENCE-ED-D

GATE SCENARIO

Can the lock be opened to traffic in three weeks?

Status of Gate and Description

During a field trip by Corps engineers to Locks 27 on 1 Mar 89, damage to the bracing and girders on the downstream side of the upstream leaf of the lift gate for the main lock was discovered (i.e., see gate model, drawings and inspection summary). The gate was out of the water due to repairs being made to the wall and corner protection of the partially unwatered lock. A more indepth inspection of the lift gate on 9 Mar 89 revealed that the damage to the gate was serious, and that immediate repairs might be necessary. Engineering assessment on the gate conditions was recommended.

The repairs will have to consider the time restrictions primarily due to the economic impact caused by the closing of the locks. A maximum of three weeks period is considered acceptable under the current circumstances, however, permanent structural repairs usually require several months to complete. Temporary repairs of the critical structural members are recommended in order to reopen the locks in three weeks.

Engineering decisions are sought to assess the structural integrity of the life gate containing cracks in the bracing members and girder flanges. Critical structural members need to be identified and a temporary repair procedure needs to be developed. In addition to the temporary solution to the damaged gate, approaches to a long-term solution on permanent repairs and post-repair maintenance schedules are also sought.

G-2

FACILITATOR GUIDE

GATE

1. Formlate and engineering approach to both a temporary and a permanent solution to the gate problem. The method of investigation should cover the following areas:

a. Inspection methods to determine the extent of cracking.

b. Cause of problems (e.g., design deficiency and/or operational deficiency).

c. Type of field investigations appropriate.

d. Type of anaiysis appropriate.

e. Repair procedures or repair alternatives.

f. Basis for recommendations.

2. Present final recommendations with a soun basis (i.e., engineering assumptions) repairs and the permanent solutions to the gate problems, respectively.

Temporary Repairs

a. Is the immediate repair to the current gate conditions required? If not, explain your reasons.

b. Can the lock be opened to traffic in three (3) weeks if temporary repairs are necessary?

c. Develop a repair plan, recommended repair schedule, resources requirement and quality assurance program.

d. What are your recommended repair alternatives? Any Codes or Specifications you would like to use to develop the repair procedures?

e. Are temporary repairs sufficient to maintain the gate integrity for the next two years? Do you recommend any scheduled inspections and assessments? If yes, what is the basis for that schedule?

Permanent Solution

a. What would be your recommended long-tern solution to the gate problem? (e.g., permanent structural repairs must be completed within a two-year period).

b. Do you recommend any design revisions to weld connections? What is the basis of your recommendations?

c. Do you recommend material tests? If yes, explain the significance of these material properties.

d. Do you recommend any further investigation/analysis to assess the current gate design and operation conditions? If yes, what should be done?

e. Any alternatives that you would like to recommend?

3. Attach calculations (if any).

4. Additional Material Available if Requested:

a. FEM analysis and strain gage test summary.

b. Materials test data summary.

c. Crack growth predictions (4 cases).

d. "Fracture and Fatigue Control", Textbook by Barsoin & Rolfe.

e. "Structural Inspection and Evaluation of Existing Look-Gates", Draft ETL, U.S. Army Corps of Engineers.

f. Inspection results of the auxiliary lock.

G-4

REFERENCE MATERIALS

GATE

The following is a list of materials available for your review and may be used for the structural assessment:

a. Gate model.

b. Actual photographs of gates and damage.

c. Design Drawings.

d. Specifications.

e. Perspective showing loading conditions (3-D sketch).

f. Inspection reports.

g. Miscellaneous data.

h. AWS D1.1 Structural Welding Code-Steel.

i. AISC Construction Manual.

j. U.S. Army Corps of Engineers EM 1110-1-2101.

k. Instructions for Operation and Maintenance of Locks No. 27.

l. Accident Summary, Locks No. 27.

m. Others may be available upon request.

CHALLENGE WORKSHOP REPORT

GATE PROBLEM, TEAM 1

1a. What must be done, if anything, to open the lock in 3 weeks as scheduled? PERFORM VISUAL LIQUID DYE PENETRANT INSPECTION. IDENTIFY DAMAGED MEMBERS AND EXTENT OF CRACKING. DURING REPAIR, METHODS TO RELIEVE LOAD TO EXTENT POSSIBLE. DRILL HOLES AT POINT OF CRACK TO RELIEVE PROPAGATION OF CRACKS WHERE APPLICABLE. INCREASE MEMBER SIZE OF TENSION MEMBERS TO REDUCE STRESS. IDENTIFY 2 OR 3 DIFFERENT TYPES OF CONNECTION REPAIR METHODS, INSTALL COVER PLATES TO GIRDER FLANGES ACROSS CRACKED SECTION AND PROVIDE WELD SIZE TO DEVELOPE FULL STRENGTH OF THE MEMBER USING QUALIFIED WELDER AND PROPER WELDING PROCEDURES. INSTALL INSTRUMENTATION (STRAIN GAGES, LOAD CARRYING MEMBERS TO PROVIDE FOR LATER ANALYSIS.

1b. What is the basis for this decision?

TIME CONSTRAINTS LIMIT ANALYSIS AND EXTENSIVENESS OF REPAIRS. THEREFORE, CONVENTIONAL METHOD OF REPAIR IS USED. GUSSETT PLATES WILL DISTRIBUTE STRESS CONCENTRATIONS THAT ADDED TO THE TENDENC OF CRACKS, TO FORM AT THE CONNECTION COVER PLATES WILL STRENGTHEN EXISTING MEMBERS.

2a. Are additional investigations and repairs needed, within 2 years, to ensure long term adequacy of the gate? WE RECOMMEND THAT A IN-DEPTH INVESTIGATION* BE CONDUCTED ON THE EXISTING STRUCTURE. THIS SHOU INCLUDE: IMPLEMENTATION OF THE INSTRUMENTATION INSTALLED AT THE TIME O REPAIR; A 3-D MODEL OF THE FORCES ACTING ON THE STRUCTURE. THE INVESTIGATION SHOULD TRY TO IDENTIFY PROBLEMS IN THE ORIGINAL DESIGN AND THE MODE OF FAILURE. THIS INFORMATION SHOULD BE USED TO DEVELOPE A NEW GATE DESIGN. * = INCLUDING ULTRA-SONIC TESTING OR OTHER NDT METHODS.

2b. What is the basis for this decision? BASED ON CRACKING PATTERNS AND FAILURE MODE OF THE EXISTING GATE, IT IS OBVIOUS THAT THE PRESENT DESIGN IS INADEQUATE. CONSIDERATION SHOULD BE GIVEN TO THE FEASIBILITY OF REPLACING THE EXISTING GATE WITH A NEW GATE AND REHABILITATING THE EXISTING GATE TO BE USED AS A SPARE GATE. THE RESULTS OF THE STUDY IN 2a. ARE NEEDED TO DETERMINE THIS FEASIBILITY.

2c. IN SUMMARY, RECOMMENDATION: DESIGN A NEW GATE, FABRICATE AND INSTA IN 2 YEARS. IF ANALYSIS SHOWS EXISTING GATE TO BE BASICALLY SOUND, IT COULD BE MODIFIED, REPAIRED (Use additional sheets if needed.) AND USED AS A SPARE.

G-6

CHALLENGE WORKSHOP REPORT

GATE PROBLEM, TEAM 2

1a. What must be done, if anything, to open the lock in 3 weeks as scheduled?

(1) DEWATER AND LOWER GATE TO SILL FOR TEMPORARY REPAIR BY MAINTAINANCE CREW; PROVIDE ADEQUATE ENVIRONMENT FOR WELDING, AND

(2) REPAIR FLOATATION CHAMBERS AND RE-ESTABLISH AIR LINES.

PRIORITY (3) ADD SPLICE PLATES TO FULLY FRACTURED BRACE MEMBERS AND FLANGES WITHOUT COVER PLATES. FULL PEN REPA OF FLANGES WITH COVER PLATES. FOR PARTIALLY CRAC MEMBERS ADD COVER PLATES, EXCEPT WHERE FULL PEN REPAIR REQUIRING LESS EFFORT ADD GUSSETS ON CRITICAL JOINTS.

1b. What is the basis for this decision?

BOYANCY LOSS CAUSED EXCESS (57%) STRESSES CLOSE TO THE YIELD; FOLLOWED BY BARGE IMPACT CONTRIBUTING TO VERTICAL LOAD WHICH PROBABLY INITIATED THE FAILURE OF THE BRACING. THE FLANGE APPEARS TO BE A SECONDARY FAILURE, DUE TO DEADLOAD INITIATING CRACKS FROM THE BOTTOM OF THE FLANGE UPWARD.

2a. Are additional investigations and repairs needed, within 2 years, to ensure long term adequacy of the gate?

YES.
- COMPLETE THOROUGH ASSESSMENT OF DAMAGE (...
- MAKE ADDITIONAL ANALYSIS WITH FINITE ELEMENT CODE.
- COMPAIR COST OF REPAIR VS. REPLACEMENT
- EXPERT EVALUATION OF WELDMENT IN EXIST DESIGN.
- ACTION BASED ON STUDIES

2b. What is the basis for this decision?

INADEQUATE TIME FOR COMPLETE REPAIR PLAN FOR PERMANANT FIX.

(Use additional sheets if needed.)

G-7

CHALLENGE WORKSHOP REPORT

GATE PROBLEM, TEAM 3

1a. What must be done, if anything, to open the lock in 3 weeks as scheduled? REPAIRS IN THE FORM OF WELDING UP CRACKS SHOULD BE ACCOMPLISHED WITHIN THE NEXT 3 WEEKS. [illegible] WILL BE REPAIRED WITH FULL [illegible]. DOUBLER PLATES WILL BE ADDED TO MEMBERS WITH LARGER CRACKS THE BUOYANCY CHAMBERS WILL BE DEWATERED AND INSPECTED FOR CAUSE OF LEAKAGE. REPAIRS TO THE CHAMBER WILL BE ACCOMPLISHED BASED ON FINDINGS. IF REPAIRS ARE NOT FEASIBLE AT THIS TIME, CHAMBERS MAY BE FILLED W/FOAM TO DELETE LEAKAGE INTO CHAMBER. REPAIRS WILL BE ACCOMPLISHED IN-HOUS OPERATIONS PERSONNEL WILL BE GIVEN DIRECTION AS TO LOCATION AND TYPE OF WELD REPAIR. INSTRUMENTATION WILL BE INSTALLED FOR MONITORING.

1b. What is the basis for this decision? REPAIRS MUST BE ACCOMPLIS[HED] TO ARREST THE GROWTH OF EXISTING CRACKS. PAST INSPECTION AND DAMAGE REPORTS INDICATE EXCESSIVE VIBRATION OF THE LIFT GATE UNDER VARIOUS HEAD ELEVATIONS. THIS VIBRATION COULD ACCELERAT[E] THE GROWTH OF THESE CRACKS

2a. Are additional investigations and repairs needed, within 2 years, to ensure long term adequacy of the gate? ADDITIONAL INVESTIGATI[ONS] SHOULD TO BE PERFORMED TO ENSURE LONG TERM ADEQUACY OF THE GATE. THESE INVESTIGATIONS SHOULD INCLUDE BUT NOT LIMITED TO STRUCTURAL A[ND] REVIEW OF CONSTRUCTION PROCEDURE: (I.E. QUALITY OF WELDS), AND OPERATIONA[L] PROCEDURES. BASED ON THESE INVESTIGATIONS, A DETERMINATION WILL BE MADE ON THE ADEQUACY OF THE EXISTING GATE. IF REMEDIAL ACTION IS REQUIRED THE REPLACEMENT OF THE GATE AND REPAIR OF THE GATE WILL BE COMPARED WITH A LIFE CYCLE COST ANALYSIS.

2b. What is the basis for this decision? FURTHER INVESTIGATION IS REQUIRED TO ENSURE A COMPLETE SOLUTION TO THE PROBLEM.

(Use additional sheets if needed.)

G-8

CHALLENGE WORKSHOP REPORT

GATE PROBLEM, TEAM 4

1a. What must be done, if anything, to open the lock in 3 weeks as scheduled?

OPERATION { 1) LOWER THE GATES & DEWATER BUOYANCY CHAMBER, CLEAN THE BUOYANCY CHAMBER AND GATE. WELD THE HATCHES OF BUOYANCY CHA[MBER] SHUT. IMMEDIATELY INITIATE REPAIRS WITH 2A[?] OPERATION.

REPAIRS. GIRDER WEB - GRIND THE WEBS & REWELD THE WEB MATERIAL [?] CRACK FOR

GIRDER FLANGES: - GIVEN TIME GRIND THE WELD ~~[?]~~ WIDTH OF FLANGE ~~[?]~~ WELD COL[?] PLATE OVER THE FLANGE.

ANGLES & SECONDARY MEMBERS - USE GUSSET PLATE, WELD

1b. What is the basis for this decision? PICK THE CATAGORY [?]

1. CRITICAL TO THE STRUCTURE SAFETY. TO PREVENT CATASTROPHIC FAILURE

2. POTENTIAL CAUSES - FATIGUE, BUOYANCY

3. DESIGN DEFFICIENCY DUE TO THE METHODOLOGY USED AT THE TIME OF ORIGINAL DESIGN/CONSTR

2a. Are additional investigations and repairs needed, within 2 years, to ensure long term adequacy of the gate?

YES.

1) FINITE ELEMENT ANALYSIS TO SEE EXISTING STRUCTURE IS ADEQUATE OR NEED ANY ADDITIONAL REPAIRS & DETERMINE IF IT WILL BE NECESSARY TO REPLACE THE GATE.

2) INSPECTIONS DURING FUTURE DEWATERING PERIODS & EVALUATE THE PERFORMANCE.

3) CONSIDER PAINTING & OTHER COATINGS DURIN[G] SHUT OFF PERIODS.

2b. What is the basis for this decision?

1) NOT ENOUGH INFORMATION IS AVAILABLE AT PRES[ENT]

1) TO DETERMINE WHETHER TO REHAB ~~THE~~ OR REPLACE THE GATE.

2) INSPECT & TEST PERIODICALLY.

3) periodic inspection TEST during future dewatering

(Use additional sheets if needed.)

CHALLENGE WORKSHOP

EXPERT'S GATE SOLUTION

SHORT TERM REPAIRS

- DRILL HOLES AT CRACK TIPS
- WELD COVER PLATES TO CRITICAL MEMBERS
- USE QUALIFIED WELDING PROCEDURES

BASIS

- CLOSE VISUAL INSPECTION
- OBVIOUS TENSION MEMBER FAILURE
- RISK OF CATASTROPHIC FAILURE

G-10

CHALLENGE WORKSHOP

EXPERT'S GATE SOLUTION

LONG TERM OPTIONS

- REDESIGN AND REPLACE THE GATE
- MAKE EXTENSIVE REPAIRS

BASIS

- INITIAL REPAIRS DID NOT CORRECT THE ORIGINAL PROBLEM
- LIFE-CYCLE-COST DETERMINES CHOICE

G-11

CHALLENGE WORKSHOP

EXPERT'S GATE SOLUTION

INVESTIGATIONS

- CONDITION SURVEY
- NON-DESTRUCTIVE TESTING (DP & UT)
- MATERIAL TESTING
- FINITE ELEMENT ANALYSIS
- FATIGUE AND FRACTURE ANALYSIS

REPAIRS

- CHANGE JOINT DETAILS
- USE BOLTED JOINTS

G-12

FACILITATOR REPORT

FACILITATOR Henry Stewart TEAM 1

1. Did this team have effective leadership? Yes ✓ No___

Comments: Although no team me had too much gate experience, th problem was handled adequately

2. Were the leaders:
   - ✓ Volunteers
   - ___ Selected by vote
   - ___ Appointed

Comments: Team Leader: Eric Sampson. Recorder: William Strobach.

3. Did the team members:

		All	Most	Few	None
a.	Participate in problem solving?	✓			
b.	Understand gate/bridge design?		✓		
c.	Understand inspection/evaluation?		✓		
d.	Understand fatigue/fracture concepts?			✓	

Comments:

4. Did the team work as:
   - ✓ Single group
   - ✓ Subgroups
   - ___ Individuals

Comments:

5. Did the team develop a work plan for attacking the problem?
   - ✓ Yes, an effective plan
   - ___ Yes, but less effective
   - ___ No

Comments: Within the time frame allocated to solve the problems, work plan was handled professionally.

6. Did the team develop adequate recommendations? Yes ✓ No___

Comments:

7. What were the recommendations? (Attach the team's report.)

8. Additional observations by facilitator:

G-13

FACILITATOR REPORT

FACILITATOR TOM RUF. TEAM 2

1. Did this team have effective leadership? Yes X No___ Comments: Leader did a good job of guiding group.

2. Were the leaders: Comments:
   - X Volunteers → Recorder Volunteered
   - ___ Selected by vote
   - ___ Appointed
   - X Flipped Coin For Leader

3. Did the team members:

		All	Most	Few	None
a.	Participate in problem solving?	X			
b.	Understand gate/bridge design?		X		
c.	Understand inspection/evaluation?		X		
d.	Understand fatigue/fracture concepts?		~~X~~	X	

   Comments:

4. Did the team work as: Comments: Worked together effectively as a group. Everyone contributed
   - X Single group
   - ___ Subgroups
   - ___ Individuals

5. Did the team develop a work plan for attacking the problem? Comments: Addressed long term + short term solutions.
   - ~~X~~ Yes, an effective plan
   - X Yes, but less effective
   - ___ No

6. Did the team develop adequate recommendations? Yes X No___ Comments:

7. What were the recommendations? (Attach the team's report.)
   Perform emergency temporary repairs. Contact expert. Do long term, in-depth analysis Perform cost analysis.

8. Additional observations by facilitator:
   Members all seemed interested in problem and enjoyed participating in workshop.

G-14

FACILITATOR REPORT

FACILITATOR *JOE HARTMAN* TEAM *GATE #3*

1. Did this team have effective leadership? Yes ✓ No___

   Comments:

2. Were the leaders:
   - ✓ Volunteers
   - ___ Selected by vote
   - ___ Appointed

   Comments:

3. Did the team members:

	All	Most	Few	None	
a. Participate in problem solving?		✓			
b. Understand gate/bridge design?		✓			
c. Understand inspection/evaluation?			✓		[illegible]
d. Understand fatigue/fracture concepts?			✓		Vague

[illegible]

Comments:

4. Did the team work as:
   - ✓ Single group
   - ___ Subgroups
   - ___ Individuals

   Comments:

5. Did the team develop a work plan for attacking the problem?
   - ___ Yes, an effective plan
   - ✓ Yes, but less effective
   - ✓ No

   Comments: *Just started talking about ideas. But some were reading*

6. Did the team develop adequate recommendations? Yes ✓ No___

   Comments: *Needed a little prodding to get on the right path*

7. What were the recommendations? (Attach the team's report.)

   Most thought it was overload, didn't recognize poor details

8. Additional observations by facilitator:

   Considered using heat to [illegible] compress welds!

   " repairing bouyancy chambers

   Consulted Operations about what crews could accomplish.

G-15

FACILITATOR REPORT

FACILITATOR Rolf. Kelsey TEAM Gate # 4

1. Did this team have effective leadership? Yes X No___

Comments: leader took charge very well. solicited ideas from team membe

2. Were the leaders:
   - ____ Volunteers
   - X Selected by vote
   - ____ Appointed

Comments: No volunteers. Individual selected had suggested a vote meth

3. Did the team members:

		All	Most	Few	None
a.	Participate in problem solving?		X		
b.	Understand gate/bridge design?		(X	X)	
c.	Understand inspection/evaluation?		X		
d.	Understand fatigue/fracture concepts?			X	

Comments: most participated to some degree. Had to explain ~~the~~ operation of locks to one individual. Very few familiar with AISC fatigue design.. Most were more familiar with concrete design rather than steel design.

4. Did the team work as:
   - X Single group
   - ____ Subgroups
   - ____ Individuals

Comments: Team worked well together

5. Did the team develop a work plan for attacking the problem?
   - X Yes, an effective plan
   - ____ Yes, but less effective
   - ____ No

Comments: Very few items left out from "expert" solution (Had to be "prodded" into developing a ~~[illegible]~~ plan)

6. Did the team develop adequate recommendations? Yes X No__

Comments: for the most part followed the "expert" solution

7. What were the recommendations? (Attach the team's report.)

8. Additional observations by facilitator:

Did not have enough time to adequately plan a long term solution. Lots of reference material. Really too much to go thru in 3 hrs.

G-16

TEAM RECOMMENDATIONS FOR GATE PROBLEM

1. SHORT TERM REPAIRS. (Where more than one team made a similar recommendation, the numer of teams is shown in parenthesis after that recommendation.)

Find additional cracks using dye penetrant inspection.

Drill holes at crack tips to halt crack growth.

Relieve loads on the gate prior to weld repairs.

Increase bracing sizes during repairs.

Weld cover plates across most cracks. (4)

Use qualified welding procedures. (2)

Install strain gages to monitor gate. (2)

Repair bouyancy chambers to relieve some loads. (2)

Use full penetration welds to repair some cracks. (3)

Check new welds with ultrasonic testing.

2. BASIS FOR SHORT TERM REPAIRS.

Crack growth rate could accellerate.

There is danger of complete failure if no repairs are made.

Ineffective bouyancy chamber causes higher member stresses.

There is insufficient time for an adequate study.

3. EVALUATION OF SHORT TERM REPAIR RECOMMENDATIONS.

Most of the team recommendations are very good. The most important of these are: Drill hoes at crack tips, weld cover plates across cracks, use qualified welding procedures, repair the bouyancy chambers. However, there is one poor recommendation; using full penetration welds to repair some cracks. Obtaining a good quality full penetration weld during field repairs is nearly impossible. The cracks are irregular, and edge preparation would be difficult. This type of weld repair can also result in very high residual stresses since the gate structure restrains the weld and resists shrinkage during post-weld cooling. Such welds have been known to crack even before all repairs are completed.

4. LONG TERM RECOMMENDATIONS.

Perform non-destructive testing to find all cracks. (2)

Add gate instrumentation to check stresses.

Perform a finite element analysis. (3)

Study the mode of failure at the cracks.

Review original design calculations.

Design a new gate and use the existing gate as a spare.

Compare costs of repairs versus a new gate. (2)

Evaluate quality of existing welds. (2)

Review operations procedures for the gate.

Conduct interim inspections after short term repairs.

Add corrosion coatings during future lock closures.

5. BASIS FOR LONG TERM RECOMMENDATIONS.

Short term repairs are not adequate as a permanent solution.

The short term evaluation was not adequate to identify all causes of the cracking.

The existing design is obviously inadequate.

6. EVALUATION OF LONG TERM RECOMMENDATIONS.

Most team recommendations are appropriate. Additional work is necessary to ensure long term gate adequacy since short term repairs did not correct (or even identify) all of the original problems. The first step in determining an adequate solution is to understand causes of the cracking. Studies to determine the causes should include: review of the original design, review of operational history and procedures, finite element analysis to better understand stresses, material testing (including tests for toughness such a Charpy V-notch tests), fatigue and fracture analysis.

These studies may indicate whether the gate is repairable to ensure long term performance. They may also indicate what revi-

sions in gate design are necessary during rehabilitation or during design of a new gate. The choice between repair or replacement of the gate should be based on a comparison of life-cycle-costs. Selection of the repair option would be subject to a thorough condition survey of the gate, including non-destructive testing to detect any additional cracks.

For the real gate the following causes of cracking were identified: poor detailing of joints resulted in high stress concentrations, the joint details were poor for fatigue life, fatigue was ignored during design, the steel had low toughness, several barge impacts overloaded the gate, bouyancy chamber malfunctions overloaded the gate, certain operating conditions were not considered during design, hand calculations used for original design did not identify all stresses predicted by finite element analysis. Due to the age and condition of the gate, and since some of the above causes were difficult to eliminate, the real gate is being replaced rather than repaired.

BRIDGE

BRIDGE

BRIDGE TEAM 1	William Wigner - Facilitator	
	James M. Ryan	CENPS-EN-DB-SD
	Raymond Veselka	CESWD-ED-TS
	David R. Descoteaux	CENED-ED-DG
	Nathan M. Kathir	CENCS-ED-D
	Stacey C. Anastos	CENAD-EN-TS
BRIDGE TEAM 2	Tom Mudd - Facilitator	
	Anjana K. Chudgar	CEORD-PE-TS
	Rick Lambert	CESAC-EN-DA
	Ted Solano	CESWA-ED-TG
	Peter Rossbach	CENAB-EN-D
	Roland Chong	CEPOD-ED-DA
	David J. Smith	CEMRD-ED-DF
BRIDGE TEAM 3	Cameron Chasten - Facilitator	
	John Burnworth	CELMK-ED-DN
	C. J. Patel	CEORP-ED-DS
	Kirti S. Joshi	CESAS-EN-DS
	Carl Mertz	CESWG-ED-DS
	John White	CESPK-ED-A-JC
BRIDGE TEAM 4	Ray Dewey - Facilitator	
	Joe Schmidt	CENCD-PE-ED-TT
	William Wallace	CESWF-ED-DT
	Jack Granade	CESAM-EN-DG
	Don Bergner	CESPD-ED-TJ
	Terry Cox	CELMV-ED-TS
	Ken Ning Chin	CENAN-EN-DR

B-1

BRIDGE SCENARIO

(1) -Bridge system has 14 simple-supported twin spans for eastbound and westbound traffic

-Bridge span: 113'-6'' c.c. bearing

-Each bridge has three 12 ft. wide traffic lanes: 40 ft. curb to curb minimum

-Each bridge has six W36 x 230 beams at 8'-0'' spacing

-Each beam has two welded coverplates at the bottom flange. Termination of primary coverplate at 9'-3'' from bearing; of secondary coverplate at 18'-3'' from bearing. There is also a coverplate on the top flange.

-R. C. deck 7-1/4 in deep composite with steel girders. Bituminous overlay 2 in. thick, about 11-1/2 years after opening.

-Concrete strength; 4500 psi
Steel: A242 with yield point of 42 ksi

(2) -About 13 years after opening to traffic, a crack was found in one of the spans in the eastbound bridge. The crack was discovered during inspection of repainting workmanship.

-Crack was in girder No. 4, at the west end of primary coverplate through the tension flange and 16 in. into the web. The crack originated at the toe of the transverse fillet weld.

-Girder No. 5 also was found to have fatigue cracks at the corresponding coverplate end.

-A longitudinal crack was found along the flange-to-web junction at a diaphragm connection plate.

<u>URGENT NEED</u>

NEED TO ASSESS THE SITUATION

NEED TO DEVELOP AN INSPECTION AND REPAIR PROCEDURE

B-2

CHALLENGE WORKSHOP

BRIDGE PROBLEMS

FACILITATOR GUIDE

1. Method of investigation should cover the following areas

   a. Cause of problem (structural detail deficiency, high volume of traffic)
   b. Type of field inspection for cracks
   c. Type of analysis for fatigue life evaluation
   d. Types of repair and procedure
   e. Basis for decision and recommendations

2. Immediate action and recommendation

   a. Can two lanes handle the traffic volume if one lane is closed?
   b. Posting for reduced load?
   c. How long will it take to repair the fractured girder (time for design of connection, fabrication and installation)?
   d. Where to inspect and how to inspect (all ends of coverplates and all diaphragm connection plates? Visual, dye-penetrant, or other procedure?)
   e. Could the adjacent girder with crack suddenly fracture (material toughness and critical crack size)?
   f. Would the longitudinal crack cause fracture of girder?
   g. Is the repair of fracture temporary or permanent?

3. Long term action and recommendations

   a. What is the estimated fatigue life of the coverplate ends?
   b. What procedure or method to determine the stresses at the ends of coverplates?
   c. What procedure or method to repair relatively large cracks (not yet fractured)? Small cracks?
   d. What recommendations for evaluating the longitudinal crack at diaphragm?
   e. How often to inspect? What's the basis for decision?

4. Attach any calculation

REFERENCE MATERIAL

BRIDGE

The following is a list of material available for your review and may be used for the structural assessment:

a. Plan view of bridge.

b. Section cuts of bridge.

c. Photos of bridge.

d. Photos of cracks.

e. Inspection report.

f. Design computations.

g. Specifications.

h. Traffic information.

i. AASHTO.

j. Others may be available upon request.

CHALLENGE WORKSHOP REPORT

BRIDGE PROBLEM, TEAM 1

1a. Can the bridge be open to traffic?

YES - LIMIT LOAD TO LIGHT VEHICLES - LESS THAN [illegible] TON
THIS DECISION TO BE REVIEWED AFTER [illegible] IS DONE

1b. What is the basis for this decision?

ACTUAL LOADS SHOW THAT BRIDGE IS CAPABLE OF CARRYING [illegible]
QUICK EVALUATION INDICATES THAT FATIGUE CAUSED THE [illegible] CRACK.
LIMITING THE LOAD SHOULD PREVENT THE CRACK FROM GROWING [illegible] REDUCING THE BRIDGE STRENGTH FURTHER.

2a. What investigation and repair are needed immediately? AND WILL

ADD COVER PLATES BY BOLTED CONNECTION TO FLANGE [illegible]
CHECK FOR CRACKS THRU FLANGES AT ENDS OF [illegible] IN ALL SPANS.
DRILL HOLES AT ENDS HORIZONTAL CRACK TO REDUCE STRESS CONCENTRATION.

2b. Within two (2) weeks?

PERFORM ANALYSIS TO EVALUATE SAFETY OF BRIDGE.
EVALUATE FIELD [illegible] TO DETERMINE IF BRIDGE IS [illegible] [illegible]
NEW DECISION BASED ON THESE ANALYSIS & INSPECTION.

2c. What is the basis for this decision?

WE WANT TO PERFORM AN "ENGINEERING" SOLUTION.

3a. What additional work needs to be done for long term safety and maintenance?

RESTORE STRENGTH OF FLANGES AT POINTS AT ENDS OF COVER PLATES.
METHOD TO BE WORKED OUT BY DESIGN OR [illegible] IF REQUIRED

3b. What is the basis for this decision?

EXISTING WELD DETAIL IS POOR & DOES NOT MEET CODE.

(Use additional sheets if needed.)

B-5

(from Calculation stress at end of plate ...
We think Crack is due to exceeding the allowed fatigue stress of 2.6 KSI for over 2,000,000 cycle as per ASHTO — 10.3.1A Table.

CHALLENGE WORKSHOP REPORT

BRIDGE PROBLEM, TEAM 2

1a. Can the bridge be open to traffic?

Close the middle lane with 2 cracked beams for the entire length of one side of bridge.

second Crack — see below

1b. What is the basis for this decision?

the dead load moment capacity is exceeded for a crack beam. Crack is due to exceeding fatigue stress at that

2a. What investigation and repair are needed immediately?

Repair - add flange and web covers plates.

2nd Crack - Add addition ~~coverplate~~ stiffner plate on both sides on the upper ~~flange~~ web area

2b. Within two (2) weeks?

Within two weeks all the bridges girders should be inspected, to make sure no more ~~fatigue~~ crack ~~exist~~ exist

2c. What is the basis for this decision?

because the — fatigue stress is exceeded.

3a. What additional work needs to be done for long term safety and maintenance?

- Add flange coverplates at cracked sections.
- Semi-annual inspection now and annual inspe after 2 years. Bridge safety is important

3b. What is the basis for this decision?

Exceeded fatigue stress limit at various places.

(Use additional sheets if needed.)

Second crack - in compression stress area not immediate concern. Diaphragm ... lateral ... buckl
- May want to add another stiffner

B-6

CHALLENGE WORKSHOP REPORT

BRIDGE PROBLEM, TEAM 3

1a. Can the bridge be open to traffic?

YES WITH LIMITATIONS AS FOLLOWS.
(1) MIDDLE LANE TO BE CLOSED
(2) MOVE RIGHT LANE TO SHOULDER. (3) REDUCE SPEED TO 55 MPH

1b. What is the basis for this decision?

(1) REDUCE LIVE LOAD & IMPACT. AND LOADING TO BE TRANSFFE
TO UNDAMAGED GIRDERS.

2a. What investigation and repair are needed immediately?

(1) INSPECT & REPORT ALL BRIDGE GIRDERS AT BOTTOM COVER PL
ENDS AT BOTTOM.
~~(2) DRILL A HOLE IN WEB TO LIMIT PROPAGATION CRACKS.~~

2b. Within two (2) weeks?

(1) TAKE COUPONS ~~AT ALL CRACK POINTS~~.
(2) DRILL A HOLE IN WEB TO LIMIT PROPAGATION OF CRACKS.
(3) BOLTED FLANGE SPLICE AT ALL BOTTOM FLANGE & COV. PL LOCAT
(4) PROVIDE WEB SPLICE AT CRACKED WEBS - USE BOLTED.

2c. What is the basis for this decision?

TO ~~REDUCE~~ ELIMINATE FATIGUE PROBLEM BY ~~REDUCING~~ ELIMINATING
STRESS/RANGE PROBLEM. RANGE WAS EXCEEDED BY WELDING
transversely

3a. What additional work needs to be done for long term safety and maintenance?

1. Keep a log and report crack propagation at a yearly interval.
2. Similar measures be adopted for rest of the spans.

3b. What is the basis for this decision?

To provide some measure for safety of public & property.

(Use additional sheets if needed.)

B-7

1a) Open to Traffic ?

Damage to girder? not critical

A: Partially: – close area on span 11 above beams 5 and 6

detour, that is to 2 exterior span

– restrict traffic on bridge to cars, vans and pickup.

detour trucks

– remove after temporary repairs are complete

restrict speed

will not reduce loads If detour is not allowed

1b) BASIS – This is done to keep load off of damaged members and reduce load to ~~mem~~ remaining members.

– we are now liable because we know of cracks in beams.

& professional obligation

B-8

2a) Immediate Investigation and Repair

A: 1) Repair cracks:
- drill hole at top of crack in web
- bolt on cover PL's on both sides of webs and flanges in area of cracks on beams 4 and 5

2) Inspect deck and remaining beams:

- Use painters equipment.

- send structural engineer out to accompany painters on inspection
  - look for dips in deck
  - look for cracks in flange or cover plates

Basis:
- stop any further cracking of 4 and 5
- determine if any other beams are cracked

B-9

2b) Investigation / Repair in 2 weeks —

A: 1) contact original designer

2) review plans / specs and design

3) Perform material tests weld & base metal (NDT) ultrasonic

verify A 242 steel for beam and, verify welds.

~~* NOTE: WELD AT END OF COVER PL IS NOT REQ'D.~~

4) perform full inspection in accordan with present day AASHTO standar

- review design.
- inspect substructure / superstructu
- map all cracks.

use visual inspection / dye penetratio. tests ~~and others~~

2c) ~~BASIS~~: to determine what caused cracks and why they occured.

B-10

3a Long Term Safety and Maintenance

A: 1) Perform periodic inspections in accordance with present AASHTO standards

2) On the basis of full inspection, effect major repairs, if any.

3b) BASIS:
- Unsure of design
- good practice to perform 2 year inspection.

B-11

Challenge Workshop

Bridge Fatigue Problem Solution

1. • Bridge has six or more girders supporting a deck. Fatigue cracking and fracturing of one girder did not result in collapse of the bridge, because other girders shared the duty of the fractured girder: Redundancy. Therefore, bridge could be kept open to traffic. Conservatively, the lane directly above the fractured girder was closed so as to reduce direct loads on this girder and on girder no. 5, which was found to have a crack.

2. • Immediate repair of the fractured girder was necessary. Repair by splicing the bottom flange and the web, assuring a field splice for design of the joint. (If well organized, this repair could be done within three days of discovery of fracture).

   • It was important to inspect the entire bridge immediately for fractured girder or girders with large cracks at the ends of coverplates.

   • A more thorough inspection of these details should be done within two weeks. The fracture of one girder and cracking of the neighboring girder suggested that the fatigue life of all coverplate ends could have been exhausted, or close to being exhausted. (The fatigue strength of welded, coverplates is designed by Category E or E' of AASHTO design specifications).

   • After splicing the fractured girder, and there was no other fracture, the closed traffic lane could be open.

   • If desired, the fractured surface of the failed girder could be removed for in depth examination in laboratory.

3. • Depending on the results of inspection during the two weeks after discovery of fracture, large size cracks could be deactivated by splicing the flange. Many cracks were detected, from very small (barely visible) to quite large (more than halfway across the flange width). Splicing of all cracked locations was too costly.

   • Evaluation of fatigue crack growth and sudden brittle fracture was needed. Examination of ADTT revealed that about 25 million trucks crossed the bridge in the thirteen years. This generated more than 25 million cycles of stresses and pretty much exhausted the fatigue life of some of the coverplate ends. If crack growth were moderate or slow, and sudden fracture would occur only when a crack was almost through the entire flange width, then large cracks could be spliced as they develop.

   • Continued inspection of the bridge was needed. Semi-annual inspection to monitor cracks and to detect new cracks was recommended.

   • Nondestructive inspection for smaller cracks and trial repair by peening or rewelding of welds were also recommended.

- The longitudinal crack at the diaphragm connection plate was due to out-of-plane displacement of the web, caused by the diaphragm action and relatively very rigid deck and top flange. Since the crack was in the direction of the primary bending stress of the web, this bending stress had little effect on the crack. As the crack grew longer, it would relieve the constraint to out-of-plane movement and the out-of-plane bending stresses in the web would be reduced. Observation of the growth of this crack was recommended.

FACILITATOR REPORT

FACILITATOR William Wagner TEAM BRIDGE TEAM 1

1. Did this team have effective leadership? Yes X No___ Comments:

2. Were the leaders: Comments:
   - X Volunteers
   - ___ Selected by vote
   - ___ Appointed

3. Did the team members:

	All	Most	Few	None
a. Participate in problem solving?	X			
b. Understand gate/bridge design?	X	X		
c. Understand inspection/evaluation?			X	
d. Understand fatigue/fracture concepts?			X	

Comments: FEW RECOMMENDATIONS CAME FORTH CONCERNING PERIODIC INSPECTION

4. Did the team work as: Comments:
   - X Single group
   - ___ Subgroups
   - ___ Individuals

5. Did the team develop a work plan for attacking the problem? Comments: GROUP DID NOT CONCENTRATE ON FRACTURE CRITICAL METHODS.
   - ___ Yes, an effective plan
   - X Yes, but less effective
   - ___ No

6. Did the team develop adequate recommendations? Yes X No___ Comments:

7. What were the recommendations? (Attach the team's report.)

8. Additional observations by facilitator:
UNDERSTANDING OF FATIGUE DEFINITELY A PROBLEM.

B-14

FACILITATOR REPORT

FACILITATOR Tom Mudd TEAM 2

1. Did this team have effective leadership? Yes ✓ No ___

   Comments:

2. Were the leaders:
   - ✓ Volunteers
   - ___ Selected by vote
   - ___ Appointed

   Comments:

3. Did the team members:

		All	Most	Few	None
a.	Participate in problem solving?	✓			
b.	Understand gate/bridge design?			✓	
c.	Understand inspection/evaluation?			✓	
d.	Understand fatigue/fracture concepts?			✓	

   Comments:

4. Did the team work as:
   - ✓ Single group
   - ___ Subgroups
   - ___ Individuals

   Comments:

5. Did the team develop a work plan for attacking the problem?
   - ___ Yes, an effective plan
   - ✓ Yes, but less effective
   - ___ No

   Comments:

6. Did the team develop adequate recommendations? Yes ✓ No ___

   Comments: Emergency procedures in ~~recommending~~ opening the bridge w/ middle span closed, track alarm, w the open procedure

7. What were the recommendations? (Attach the team's report.)

8. Additional observations by facilitator:

These team members did not have much bridge design or evaluation experience, however by using their professional judgement and problem solving techniques they did an excellent job in analyzing the problem, designing solutions and making [illegible]

B-15

FACILITATOR REPORT

FACILITATOR Cameron Chasten TEAM BRIDGE TEAM 3

1. Did this team have effective leadership? Yes X No___

Comments: The "leader" & one other shared leadership.

2. Were the leaders:
   - X Volunteers
   - X Selected by vote
   - ___ Appointed

Comments: one person with bridge experie ~~one~~ "volunteered", with some coaxing for leader. Recorder was "Volunteered" also.

3. Did the team members:

		All	Most	Few	None
a.	Participate in problem solving?		X		
b.	Understand gate/bridge design?			X	
c.	Understand inspection/evaluation?				?
d.	Understand fatigue/fracture concepts?			X	

Comments: Not enough time to really talk about inspection. Bridge or fatigue was not understood. They did with some guidance.

4. Did the team work as:
   - X Single group
   - ___ Subgroups
   - ___ Individuals

Comments: Team effort defintely, one person was very quiet, everyo else equally effective.

5. Did the team develop a work plan for attacking the problem?
   - ___ Yes, an effective plan
   - X Yes, but less effective
   - ___ No

Comments: The plan basically was map by the questions much time spent on ~~to~~ quest 1 and 2.

6. Did the team develop adequate recommendations? Yes X No___

Comments: The team ~~[illegible]~~ needed guidance to understand th fatigue problems details. Once th understood, ~~[illegible]~~ recomendat. adequa

7. What were the recommendations? (Attach the team's report.)

8. Additional observations by facilitator:

Team was baffled by insignificant details, i.e. material Chemical properties, web longitudinal crack.

B-16

FACILITATOR REPORT

FACILITATOR Ray Dewey TEAM Bridge - 4

1. Did this team have effective leadership? Yes x No___

Comments: It was shared team, but seemed to get at all answers, not necessarily in a sequence.

2. Were the leaders:
   - x Volunteers
   - x Selected by vote
   - ___ Appointed

Comments: Recorder volunteered quickly - Team challenged & worked for a while until I asked them to ~~volunteer~~ decide on a leader.

3. Did the team members:

		All	Most	Few	None
a.	Participate in problem solving?	x			
b.	Understand gate/bridge design?		x		
c.	Understand inspection/evaluation?			x	
d.	Understand fatigue/fracture concepts?			x	

Comments: understand design but minimal knowledge of AASHTO - but used basic understanding of structures to make up for lack of AASHTO knowledge. little knowledge of weld/cr inspection methods. Little knowledge of bridge inspection

4. Did the team work as:
   - x Single group
   - ___ Subgroups
   - x Individuals

Comments

5. Did the team develop a work plan for attacking the problem?
   - ___ Yes, an effective plan
   - ___ Yes, but less effective
   - x No

Comments: followed questions - went back & forth

6. Did the team develop adequate recommendations? Yes x No___

Comments:

7. What were the recommendations? (Attach the team's report.)

8. Additional observations by facilitator:

~~[illegible]~~

Thought that welds were causing problem

One thought fatigue was a problem.

CESEC 1991

CHALLENGE WORKSHOP SUMMARY

BRIDGE PROBLEM

DESCRIPTION OF THE PROBLEM:

The problem presented to the bridge teams, as with the gate problem, was based on an actual situation although names, location and other references were omitted in the event anyone was previously familiar with the problem. Yellow Mill Pond bridge is part of the Connecticut Turnpike and carries interstate 95 over the Yellow Mill channel. It consists of 14 consecutive simple spans; traffic flows predominately east and west with 3 lanes in each direction (see Figures 1 through 3). In order to accommodate shipping clearances in the channel beneath, cover plated W36 sections were used in lieu of deeper sections as the primary girders spanning between the piers. Double cover plates were welded on the tension flanges and single cover plates were welded on the compression flanges, all partial length.

The bridge was opened to traffic in 1958 and was repainted approximately 10 years later. In 1970, during inspection of the repainting, a crack was discovered in one of the primary girders in the eastbound side of span 11 (see Figure 4). The crack began at the toe of the weld connecting the end of the primary cover plate to the tension flange of the W36 member. It completely penetrated the tension flange and extended 16 inches into the web of the 36 inch deep beam.

The bridge teams were asked to evaluate the severity of the situation as to whether or not the bridge could remain open to traffic. They were also asked to provide recommendations for repairs and investigations that were required immediately and within two weeks (short term) and additional work that would be necessary for long term safety and maintenance (long term). The basis for all of these decisions were also requested. As an additional item, a second crack was included into the scenario (this second crack was not part of the actual problems at Yellow Mill Pond bridge but was only included as part of the Challenge Workshop). The second crack was discovered at the same time as the first in a diaphragm between two girders. Unlike the first crack, this one was a longitudinal crack located in the upper half (compression zone) of the diaphragm.

EXPERT SOLUTION:

The welded cover plate detail used at Yellow Mill Pond bridge was a category E' detail, the worst fatigue detail as classified by AISC and AASHTO. As a result, the allowable stress range was lower than it would have been had any other detail been used. This coupled with a large volume of traffic (approximately 30,000 vehicles daily, 12,000 of which are trucks) revealed that the

fatigue life of the detail had already been exceeded. However, because of the redundant nature of the multi-girder span, the load was able to shift to other members. This explained why catastrophic collapse did not occur even with a significant loss of the tension portion of the girder.

The lane above the cracked girder was immediately closed to traffic. Short term repairs consisted of bolting splice plates on the top and bottom of the tension flange of the cracked section, as shown in Figure 5. The use of bolts provides a more fatigue resistant detail than welds.

The fatigue life of a detail is determined by the type (category) of detail and the range of stress that the detail experiences. Realizing that the fatigue life of the cracked cover plate detail had been exceeded, other similar details were inspected using dye penetrant and ultrasonic testing. Figure 6 shows a crack at the end of a similar cover plate that was detected by dye penetrant. As this figure shows, these cracks are difficult to detect with the naked eye. As a result, they are often overlooked in the early stages of crack growth. Figure 7 illustrates the two stages of crack growth at the ends of welded cover plates. Crack initiation through the tension flange at the end of cover plates (Stage 1) consumes approximately 95% of the fatigue life of the detail. From there, the crack propagates quickly into the web (Stage 2) during the remaining 5% of fatigue life. As an illustration, consider a cover plated detail with a fatigue life of 50 years. For the first 47.5 years (approximate), the detail will appear similar to the one shown in Figure 6 with an almost undetectable (to the naked eye) hairline crack along the toe of the weld. During the next 2.5 years the crack will propagate rapidly, to an state similar to what is shown in Figure 4.

To ensure long term safety an annual inspection program was initiated to monitor crack growth at other cover plated sections. Repairs to cracked members using bolted cover plates were made as dictated by the inspection program.

As it turned out, the longitudinal crack found in the diaphragm was not a critical problem since it was located in the compression zone and the crack direction was parallel to the direction of stress. The structural integrity of the diaphragm was not affected due to this crack; therefore, it did not require significant treatment. Holes were drilled at the crack tips and the crack was monitored to ensure that further propagation did not occur.

TEAM ORGANIZATION:

The bridge problem presented during the Challenge Workshop required the participants to address an unfamiliar emergency situation. Unlike the gate problem where experience with steel gates was prevalent among the team members, bridge design experience among the bridge teams was minimal.

Team leaders stepped forward as volunteers. Leadership was effective, due mostly to the professional attitude taken by all members. Even though bridge experience was lacking, the team members approached the problem with enthusiasm and worked as a

single group.

PROBLEM SOLUTION:

One common observation was noted among all of the teams: while most of the members understood bridge design and AASHTO specifications, very few understood inspection/evaluation or the concepts behind fatigue/fracture. As a result, all of the groups developed a plan of attack to solve the problem that was less effective than it could have been.

TEAM RECOMMENDATIONS:

The combined recommendations of all four teams provided a solution which closely matched the expert solution. Their combined recommendations are as follows:

1a. Can the bridge be open to traffic? All of the teams concluded that the bridge could remain open, with some restrictions. These included limiting traffic to light vehicles only, closing the lane above the cracked girder, or a combination of both.

1b. What is the basis for this decision? Because the condition had existed for some time, the bridge was in no eminent danger of collapse and could remain open to traffic. Closing the lane was recommended in order to limit the load on the cracked girder.

2a. What investigations and repair are needed immediately? All of the teams recommended immediate repair of the cracked girder. Repair details varied between adding flange and web cover plates, drilling holes to limit crack propagation, or a combination of both. Two teams specified that cover plates should be bolted on (a better fatigue connection than welding).

One other action that most of the teams recommended was an immediate inspection of all similar beams and cover plates. While this idea showed good judgement, the logistics and effort involved were not practical in a short time frame. This item would be better scheduled within the next two weeks, as in the expert solution.

2b. Within two (2) weeks? Most of the groups suggested that a more detailed evaluation of the integrity of the bridge be made within the first two weeks. Action items recommended were a review of the plans, specifications, and original design, consultation with the bridge designer, evaluation of the inspection results, some non-destructive testing of other welds, and material testing on coupons taken from the cracked girder.

2c. What is the basis for this decision? Immediate actions were based on restoring the strength capacity to the cracked girder as quickly as possible and to prevent further crack propagation which could lead to catastrophic collapse. These actions were based on

conservative judgement as detailed analyses, which would dictate rehabilitative actions, could not be performed immediately.

Actions taken during the initial two weeks were concentrated on determining the cause of the crack through material testing and analytical methods and for arriving at a long term solution based on engineering analysis.

3a. What additional work needs to be done for long term safety and maintenance? Based on the results of the detailed analysis made during the first two weeks, the effectiveness of the emergency flange and web splices were evaluated and long term repairs (e.g., replacement of entire girder, new cover plates, etc.) would be made, if necessary. Also, an annual inspection program was initiated to inspect the repaired girder and other girders with similar details.

3b. What is the basis for this decision? Most teams realized that the welded cover plate was a fatigue prone detail and believed fatigue to be the cause. If so, it would only be a matter of time before cracking would occur at other cover plates. Cracks, due to fatigue, had already initiated but not to the point where they could be detected without the use of non-destructive testing (NDT) methods. The purpose of the periodic inspection was to repair fatigue prone sections before cracks propagated enough to cause an appreciable loss of section.

CONCLUSIONS:

Consideration to the effects of fatigue are often overlooked during the design and detailing of connections. While the cover plate detail used at Yellow Mill Pond bridge was acceptable, another type could have been selected which would have had a longer associated fatigue life.

The structural engineers which took part in the Challenge Workshop represented a good cross section of structural engineering expertise throughout the Corps. Better understanding of the concepts of fatigue, the principles of fracture mechanics, and NDT procedures are required by Corps structural engineers in order to effectively respond to emergency situations and the rehabilitation of distressed structures. The resourcefulness and professional approach of the Corp's structural engineers during a quick-response situation was evident among all the teams during the Challenge Workshop. The bridge workshop was not altogether meaningless for the Corps of Engineers; the Corps owns and is responsible for over 200 bridges. These include public, project related, and railroad bridges.

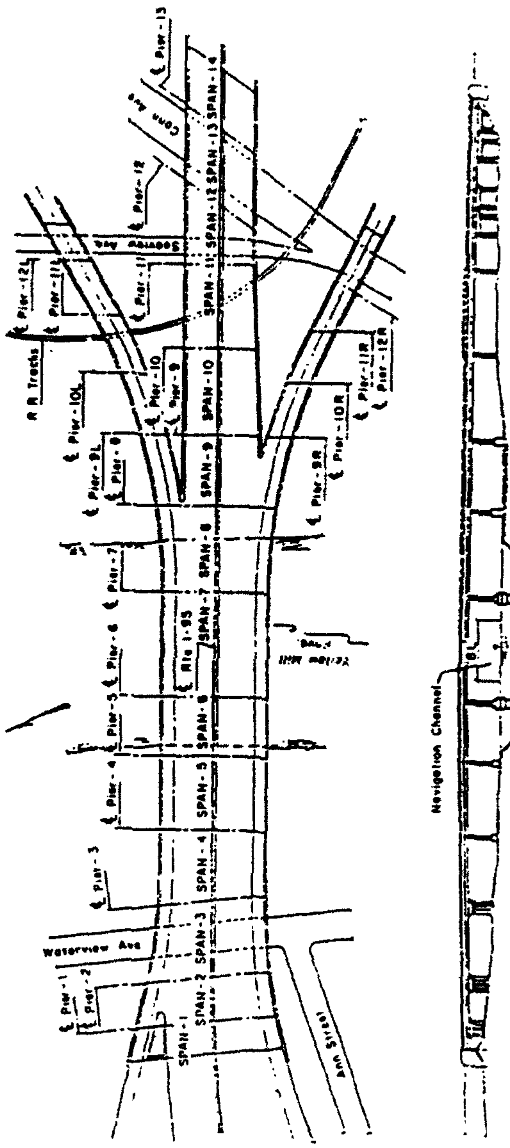


Figure 1. Plan and elevation of Yellow Mill Pond Bridge.

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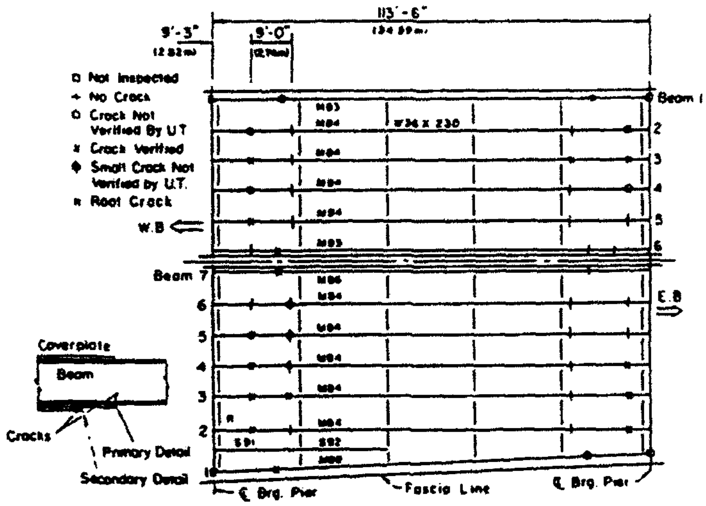

Figure 2. Plan of span 10, Yellow Mill Pond Bridge.

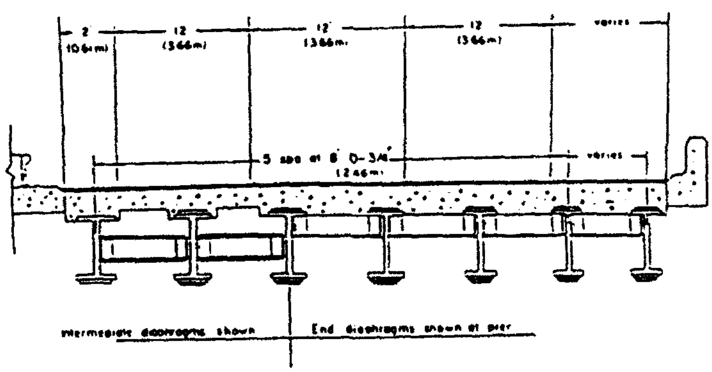

Figure 3. Typical expressway cross section.

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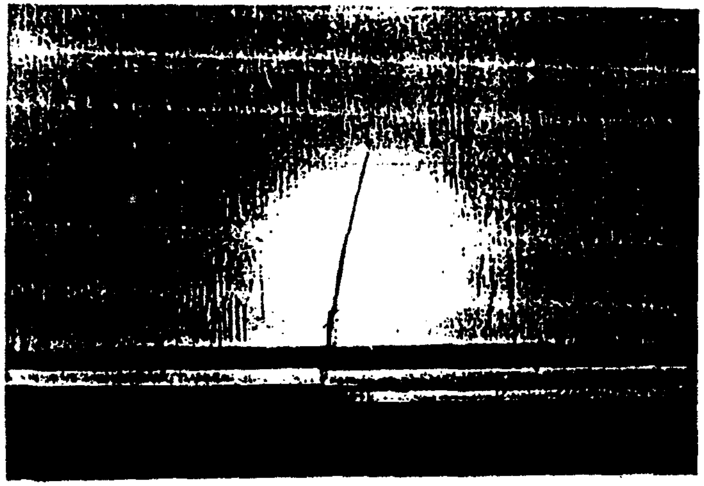

Figure 4. Cracked girder in span 11 of Yellow Mill Pond Bridge.

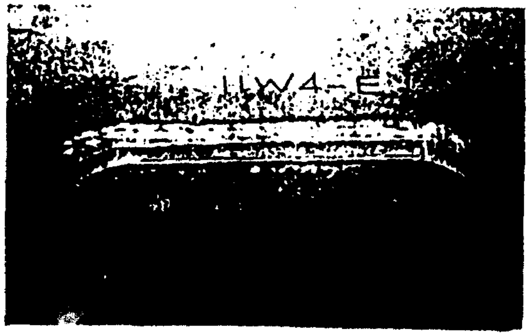

Figure 6. Typical crack at weld toe at end of cover plate.

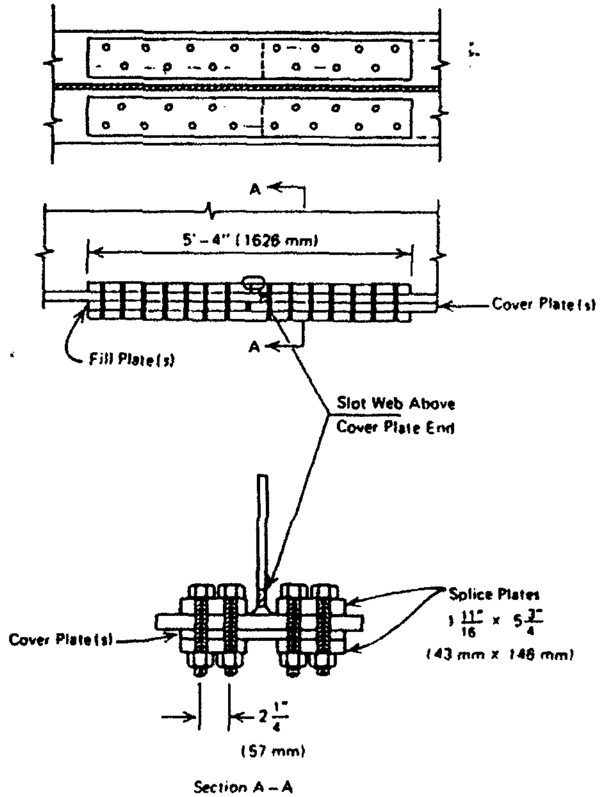

Figure 5. Retrofit details of bolted splices.

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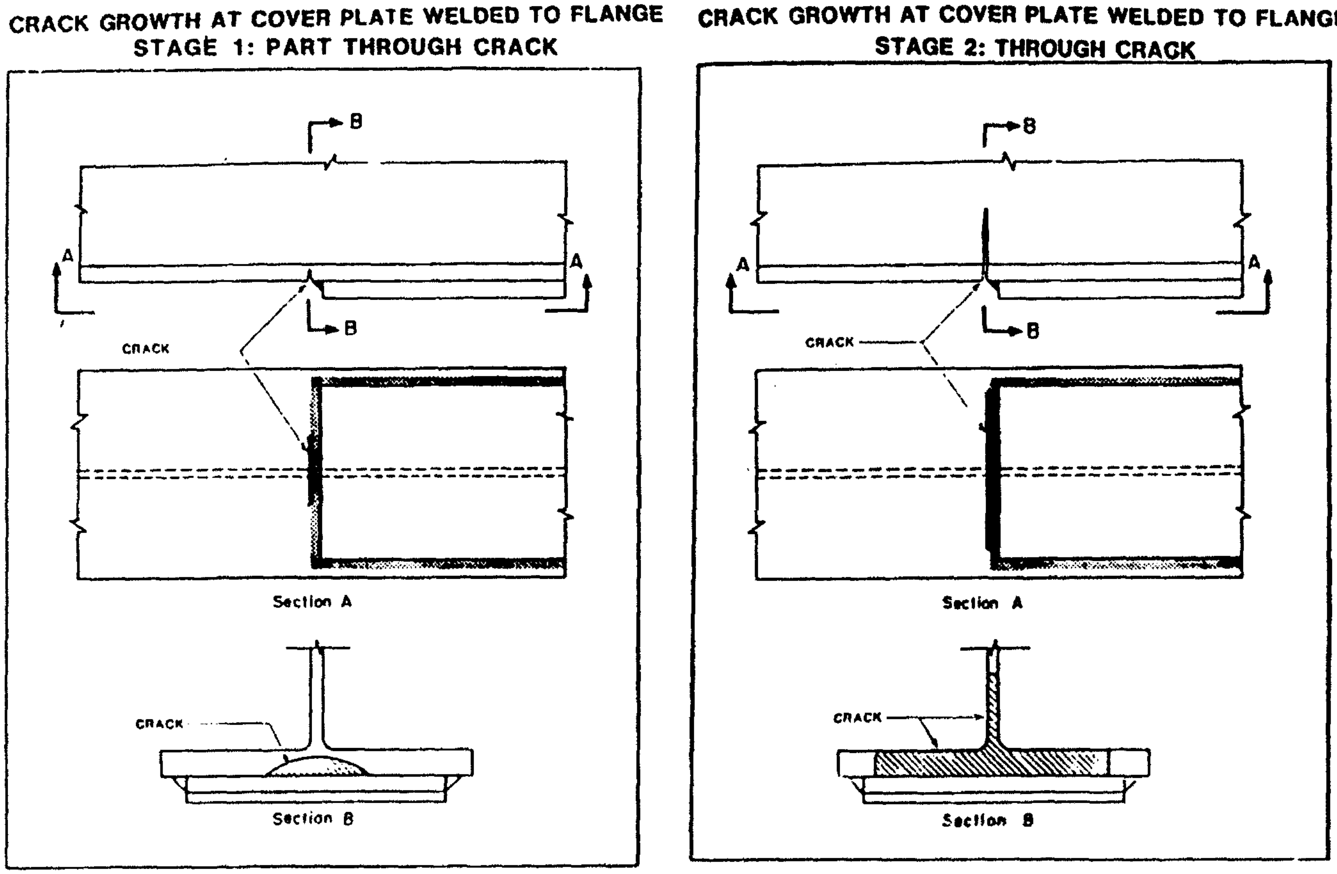

Figure 7. Stages of crack growth at ends of welded cover plates.

Quality Facility Data: Cradle to Grave

by
Ronald L. Hollrah[1]

(Copy of paper not available)

[1] Black & Veatch.

Appendix A
Conference Evaluation

U.S. ARMY CORPS OF ENGINEERS
STRUCTURAL ENGINEERING CONFERENCE

8 JULY - 12 JULY 1991

Ponte Vedre Beach, Florida

EVALUATION SUMMARY

EVALUATICN	EXCELLENT	GOOD	FAIR	NOT RATED
1. How do you rate this conference overall?	47	9	0	
2. How did you like the format for the conference sessions?	41	15	0	
3. How do you rate the quality of the oral presentations?	27	29	0	
4. How do you rate the visual aids used for these presentations?	27	27	2	
5. How do you rate the visual display/poster sessions?	19	30	2	5
6. How do you rate the Challenge Workshop?	32	14	2	8
7. How do you rate the Friday afternoon training session?	19	15	1	11
8. How do you rate the conference evening activities? (Base opinion on interest, usefulness, organization)	10	23	15	8
9. How do you rate the spouse and family activities?	9	11	7	19

TOPICS WHICH WERE THE MOST USEFUL

17	FRACTURE (STEEL/CONCRETE)
15	RETAINING AND FLOODWALL DESIGN
15	ANALYSIS/DESIGN OF STEEL STRUCTURES
13	SEISMIC ANALYSIS AND DESIGN
13	CHALLENGE WORKSHOP
10	NONLINEAR INCREMENTAL STRUCTURAL ANALYSIS (NISA)
8	DESIGN RESPONSIBILITY
8	CW AND MP LEADERSHIP FORUM
7	MAINTAINING DESIGN QUALITY
6	ANALYSIS/DESIGN OF CONCRETE STRUCTURES
6	OPPORTUNITY TO SHARE INFORMATION
6	NEW GUIDANCE FOR STRUCTURAL ENGINEERING
5	ANALYSIS/DESIGN OF MASONRY STRUCTURES
5	REHABILITATION PRESENTATIONS
5	GENERAL SESSIONS
4	STATE-OF-THE-ART PROCEDURES
3	QUALITY FACILITY DATA: CRADLE TO GRAVE
3	QUALITY IN CONSTRUCTED PROJECT
2	STRUCTURAL RELIABILITY
2	COFFERDAMS & CONSTRUCTION
2	PANEL SESSION
2	HARDENED STRUCTURES SESSIONS
1	VISUAL DISPLAY PRESENTATIONS
1	CASE PROGRAMS
1	NON-DESTRUCTIVE TESTING METHODS

COMMENT

The 1991 Corps of Engineers structural engineering conference was an outstanding success. I would like to express my sincere appreciation to those who were responsible for planning and preparing for the conference. The issues which were covered in the general sessions were pertinent and timely. The papers which were presented covered a broad range of topics, giving the participants an unparalleled opportunity to draw much knowledge from structural engineers throughout the corps. The facility which was selected was wonderful and really enhanced the experience. Thank you!

This comment submitted was typical of numerous voiced during the conference and submitted with the evaluations.

Appendix B
Civil Works Leadership Forum

Civil Works Leadership Forum

12 July 1991

Topics discussed:

LCPM/Project Managers -- Concerns
Fictitious issues
Where the money is to be spent
A/E problems
LCPM frustrations
Cost engineering separated from engineering (fragmentation)
Create win/win positions
Brokering
Engineering involvement with A/E
Upward reporting requirements of LCPM
Planning is not pushing for approved plan prior to completing feasibility report.
Assumptions and risks of incomplete design
Roles and responsibilities
Quality vs number of changes
Coordinating the technical side
Proactive
Common goals -- quality, costs, schedules
ER on engineering functions and when
Scope of effort early on
Project management for supervisors
Central data base
Should chief of project management be a PE ?
Design team approach
Performance descriptions
Quality indicators for PRB's
Position on Engineer of Record
Applicable for government agencies
Organizational changes required

List of Attendees at CW Leadership Forum

Lucian Guthrie	CECW-ED	(202)272-8673
Tom Leicht	CECW-ED	(202)272-8677
Gordon McClellan	CEORN-ED	(615)736-5023
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Byron Foster	CESAD-EN-TS	(404)331-6707
Ray Navidi	CEORH-ED-D	(304)529-5202
Bill Barnes	CEORH-ED-DS	(304)529-5217
Larry Seals	CEORD-PE-TS	(513)684-3034
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Tom Ruf	CELMS-ED-DA	(314)331-8228
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Cliff Ford	CESPL-ED-DB	FTS-798- (213)894-5530
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Richard Atkinson	CENCR-ED-DS	(309)788-6361
Tom Wright	CEMRK-ED-DT	(816)426-5172

Appendix C
Military Programs Structural Chiefs Meeting

Military Programs Structural Chiefs Meeting

9 JULY 1991
TENTATIVE AGENDA

SCHEDULE	TOPIC	SPEAKER	
0800 - 0815	Welcome and Introductions	C. Gutberlet	CEMP-E
0815 - 0900	Structural QA/QC, Engineer of Record, Responsibility for Design	P. LaHoud	CEHND
0900 - 0920	Criteria Document Update Program Overview - TMs, CEGS, 3078, CD ROM, Standard Drawings	D. Wilson	CEHND
0920 - 1020	Masonry Criteria Status	E. Staab	CEMRD
	TM, CEGS, QA/QC, Details		
1020 - 1030	Break		
1030 - 1115	Seismic Criteria Status TMs, Codes, Details 809-10, 809-10-1, 809-10-2	R. Strom	CENPD
1115 - 1200	Structural Criteria Status 809-1, 809-2, 809-4, 809-5, 809-6, 809-8	S. Wright	CEHND
1200 - 1300	Lunch		
1300 - 1400	Standing Seam Metal Roof and Metal Building Systems Criteria	L. Seals	CEORD
1400 - 1415	Break		
1415 - 1600	Open Discussion	Panel	

Revision and Update of the Basic Design Manual (BDM) "Seismic Design for Buildings"

by
Ralph W. Strom, PE[1]

Abstract

Recent changes have been made to the seismic design provisions of the various building codes. These changes affect the computation of "equivalent static forces" and the design of "lateral force-resisting systems." Equivalent static forces are used in seismic design to approximate the inertial effects experienced by buildings during an earthquake. It is uneconomical to design a building to remain elastic during a major earthquake. Therefore, the equivalent static forces are "scaled-down" forces, and dependence is placed on the lateral force-resisting systems to dissipate energy in the inelastic range. The recent code changes are being incorporated into the Tri Service Manual, TM5-809-10 (Wiss 1990), for the seismic design of buildings. These changes are the subject of this paper.

Seismic Design Codes

History

Modern seismic design codes are based on an "equivalent static force method." This method prescribes a lateral force of a given magnitude and distribution, so that application of this force to the structure approximates the inertial effects experienced by the structure during ground motions representative of the design earthquake. The code design earthquake for buildings is one that has a 90-percent probability of not being exceeded during a 50-year period. The magnitude of the lateral force is determined by a base shear formula. The distribution of this force to the various floor levels and roof is in accordance with a base shear distribution formula which assigns the higher story forces to the roof and uppermost floors. The base shear formula has changed appreciably over the past 40 years. One of the biggest changes occurred in 1976 when the base shear nearly doubled for buildings of 3 to 10 stories. Although in appearance the new base shear formula differs remarkably from that of our 1982 Tri Service Manual, the lateral force magnitude and distribution is practically the same. The lateral force provisions in this manual for the "Seismic Design of Buildings", commonly referred to as the Basic Design Manual (BDM), are based on the "Recommended Lateral Force Requirements" of the Seismology Committee of the Structural Engineers of California (SEAOC) (Seismology Committee of the Structural Engineers of California 1988). The Uniform Building Code (UBC) (International Conference of Building Officials 1988) is also based on SEAOC. The 1982 BDM and the 1979 UBC were based on the 1975 SEAOC. The new BDM and the 1988 UBC are based on the 1988 SEAOC. Many of the new SEAOC provisions stem from the Applied

[1] Structural Engineer, US Army Engineer Division, North Pacific; Portland, OR.

Technology Council's ATC 3-06 report, "Tentative Provisions for the Development of Seismic Regulations for Buildings" (Applied Technology Council 1978).

Philosophy

The design approach for load combinations involving earthquake differs from that for load combinations with wind, snow, or other live loads. The seismic code provisions allow structural members to be worked beyond their elastic limits, while the design for other loads requires that structural members remain elastic with a prescribed margin of safety based on either the yield or ultimate strength of the materials (Freeman 1979). Although the design procedure for members subject to earthquake load combinations is similar to that for members subject to other load combinations, the reason it is similar is that the seismic forces represented by the base shear formula are "scaled-down" forces. The magnitude of the force reduction is dependent on the ability of the particular lateral force-resisting systems to perform inelastically, the inherent toughness of the materials in the system, and the amount of damping associated with inelastic behavior. The scaling factor in the new code is the response modification factor R_W. The subscript "$_W$" indicates the seismic force is being scaled down to a working stress level, and the design should proceed as if this were a service load level force. The scaling factor in previous versions of the base shear formula is not readily apparent, and many designers failed to recognize that the actual force levels experienced by a structure during a major earthquake could be 5 to 10 times greater than those represented by the base shear formula. The seismic design philosophy as stated in the ATC 3-06 report (Applied Technology Council 1978) is as follows:

> "Life safety in the event of a severe earthquake is the paramount consideration in the design of buildings. With this in mind, it is intended that the provisions and principles developed by the ATC-3 project should provide an up-to-date basis for development of seismic design regulations which should enable most buildings to:
>
> 1. Resist minor earthquakes without damage.
> 2. Resist moderate earthquakes without significant structural damage, but with some nonstructural damage.
> 3. Resist major or severe earthquakes without major failure of the structural framework of the building or its component members and equipment, and to maintain life safety. It is also recognized that for certain critical facilities, particularly those essential to the public safety and well-being in case of emergency, criteria should be available to the designer which will permit design of a facility which will remain operational during and after an earthquake.
>
> It is recognized, however, that because of the random and unpredictable nature of earthquake motions and the uncertainties concerning ultimate strength capacities and the response of buildings to earthquake motions, the seismic design requirements can not fully ensure that there will be no injury or loss of life."

Total design quality

During major earthquakes, buildings can undergo several cycles of inelastic response. Good seismic designs limit yielding and plastic hinging to structural regions that will not trigger a collapse mechanism. Seismic design is much more than designing to the force level prescribed by code. Careful thought is required to ensure attainment of total design quality. This means providing:

- The best possible system with regularity in stiffness and mass distribution to avoid areas of stress concentration.

- Strong connections to keep the lateral load path system intact.
- Redundancy to prevent collapse of the structure due to collapse of a single component of the lateral force system.
- Design review, materials testing, and construction inspection to ensure the in-place structure can perform as intended during a major earthquake.

Revisions to BDM, general

As stated previously, code type revisions to the BDM will be in accordance with the 1988 SEAOC. Code type revisions not only involve changes in the lateral force provisions, but also involve changes in structural system requirements (Wiss 1990). In addition, code requirements for some structural systems not previously covered by the BDM are provided. These include eccentric braced frames and seismically isolated systems. Those familiar with the BDM recognize that this manual goes beyond code requirements and provides such useful things as:

- Background information on the development of the "equivalent static force method" from the principles of dynamics.
- Application of the code provisions to building design.
- Details, properties, and capacities of various diaphragm, shear wall, braced frame, and moment frame systems.
- Seismic protection for mechanical electrical systems.
- Seismic protection for cladding, partition walls, and other architectural features.
- Seismic design of nonbuilding structures.
- Design examples.

The revised manual will also include information on the design of foundations to resist earthquake lateral forces. Details with respect to BDM changes in lateral force requirements, system requirements, and quality control provisions are covered in the following paragraphs.

Lateral force requirements

The lateral force applied to a structure to account for earthquake effects is defined by a base shear formula. As stated earlier, this base shear represents a "scaled-down" version of the actual seismic inertial force expected during the design earthquake event. This scaling accounts for the ability of the structure to dissipate energy in the inelastic range (economic consideration) and permits an elastic analysis to be performed at service load levels compatible with that used for other nonseismic type loads. A comparison between the base shear formulas of the new BDM and the 1932 BDM is illustrated in Table 1. Both the new and old base shear formulas include parameters which account for:

- The severity of seismic ground motions at the site (Z factor).
- The dynamic amplification of motion that can occur within the structure due to the vibration characteristics of the structure and due to possible site-structure resonance (C and S factors).
- The different life safety requirements for various occupancy conditions (I factor).
- The different inherent ductility and toughness of the various lateral force-resisting systems (K and R_w factors).

Although individual parameters differ markedly between the old and new BDM, the resulting base shear for each particular seismic zone has not changed appreciably.

Seismic zones and Z factors

Seismic zone boundaries have undergone marked changes in some regions of the USA. This can be seen by comparing the old seismic zone (Figure 1) with the new map (Figure 2).

Table 1
Old and New Provisions Compared

	1982 BDM	1991 BDM
Base Shear Formula	$V = (ZICKS)W$	$V = (ZIC/R_w)W$
Seismic Zone Factor - Z	Z	Z
Zone 0	0.0	0.0
Zone 1	0.188	0.075
Zone 2	0.375	–
Zone 2A	–	0.150
Zone 2B	–	0.200
Zone 3	0.750	0.300
Zone 4	1.000	0.400
Importance factor - I	I	I
Essential facilities	1.500	1.250
High risk facilities	1.250	–
Hazardous facilities	–	1.250
Special occupancy	–	1.000
Standard occupancy	1.000	1.000
Building system factor	K	R_w
Bearing wall - concrete & CMU	1.330	6
Building frame - concrete & CMU	1.000	8
Building frame - steel braced fr.	1.000	8
Special moment resisting fr.	0.670	12
Ordinary moment resisting fr.	1.000	6
Building period	T	T
General	$0.05h_n/D^{1/2}$	$C_t(h_n)^{3/4}$
Frame	0.10N	–
Building period coefficient	C	C
Formula	$1/15T^{1/2}$	$1.25S/T^{2/3}$
Maximum value	0.12 (CSmax = 0.14)	2.75
Site coefficient	S	S
Function of T/Ts	1.0 to 1.5	–
Rock or dense soil	–	1.0
Over 200 ft dense soil	–	1.2
Soft clay	–	1.5
Over 40 ft soft clay	–	2.0
Maximum base shear	Vmax	Vmax
$\left[\begin{array}{l} T < 0.3 \text{ sec}, S = 1.5 \\ \text{Building Frame} \\ K = 1.0, R_w = 8, I = 1.0 \\ \text{Zone 4}, Z = 1.0 \ (z = 0.4) \end{array}\right]$	$1(1)(1)0.14W = 0.14W$	$\left(\frac{0.4 \times 1 \times 2.75}{8}\right)W = 0.14W$

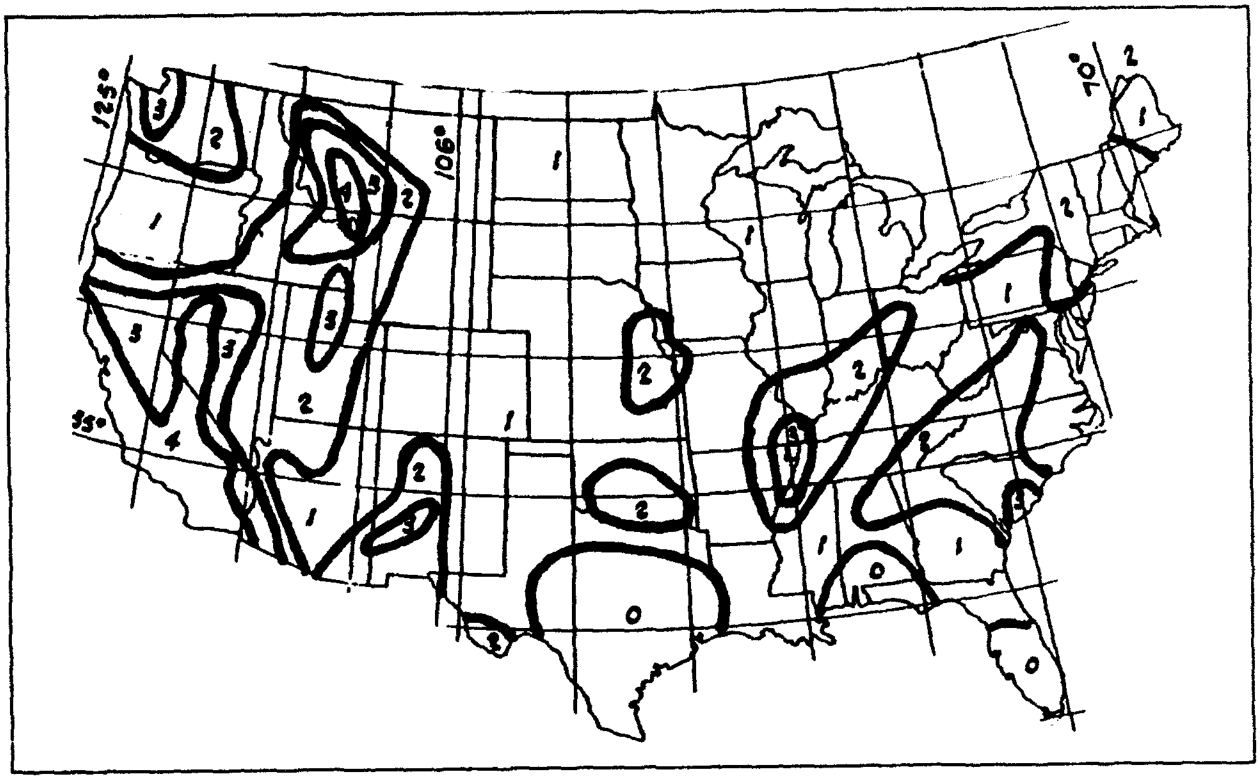

Figure 1. Seismic zone map of the USA—1982 BDM

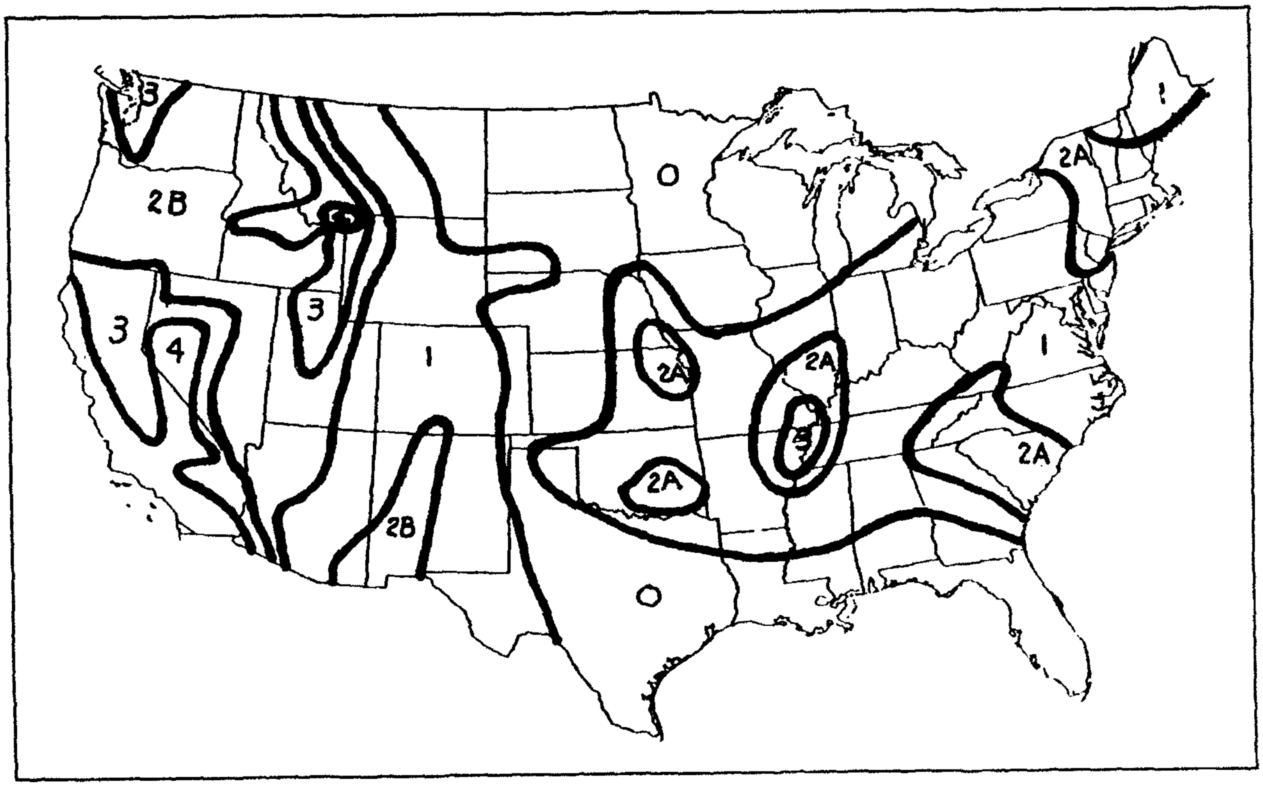

Figure 2. Seismic zone map of the USA—1991 BDM

The old zone 2 has been split into zones 2A and 2B, with zone 2A occurring in the midwestern and eastern USA, and zone 2B occurring in the western USA. Some western regions that were zone 1 have been changed to zone 2B (see Pacific NW, for example). Z values have also changed. The new Z values represent the peak ground acceleration (PGA) of the design earthquake for each seismic zone represented by the seismic map.

Dynamic amplification factor

The C value essentially represents a normalized acceleration "design" response spectra or dynamic amplification factor (DAF). C, therefore, is dependent on the fundamental period of vibration of the structure which is a function of the mass and stiffness of the structure. The dynamic amplification factor (C) is also a function of the site conditions, with largest values assigned to structures founded on soft soils. The formula for determining the fundamental period of vibration (T) has changed. In addition, the 1991 BDM will have an additional site condition factor to account for sites such as "Mexico City" which contain more than 40 ft of soft clays. A comparison between the 1982 BDM and the 1991 BDM with respect to the dynamic amplification factor and the various parameters which are part of the DAF is shown in Table 1.

Importance factor

The importance factor (I) has been included in base shear formulas to require a higher design force level for structures which house essential and hazardous facilities. This is an indirect way of keeping these facilities free from structural damage which could impair their ability to remain operational following a major earthquake. The I factor for essential buildings in the 1991 BDM is 1.25 (reduced from the 1.5 value used in the 1982 BDM). The SEAOC commentary explains the I value changes as follows:

> "The I factor for essential facilities has been reduced from 1.5 to 1.25 and new classifications of hazardous facilities and special occupancy structures have been added. The reduction in force level is made recognizing that higher force levels alone do not necessarily improve seismic performance. Experience indicates that independent design review, and appropriate program of testing and inspection and involvement of the structural engineer in the construction support process will result in a higher standard of performance. Therefore, the I factor reduction to 1.25 coupled with these new requirements are judged to achieve this purpose more reliably and economically."

Response modification factors

The base shear formula in the BDM includes a response modification factor (R_w). This factor appears in the denominator of the base shear formula and serves the same purpose as the old K factor. The R_w factor represents the ability of a particular lateral force resisting system to perform inelastically without failure. In other words, it is a measure of such things as ductility, redundancy, material toughness, and system toughness. The higher R_w values are assigned to those structural systems which rate high in these characteristics. The R_w factor, in essence, is also the scaling factor which brings the seismic force expected during the design earthquake event down to a service load or "working" stress level of design. The numerator of the new base shear formula represents the actual maximum seismic force expected during the design earthquake event assuming the structure remains elastic. The total base shear. as calculated by the base shear formula, represents a "working stress" level force. Designing for this force assumes the structure will utilize the margin of safety (working stress to yield) and inelastic performance (energy dissipated beyond yield) to resist the effects of the design earthquake event.

Building system requirements

Lateral force-resisting systems designed by the equivalent lateral force method must perform in the inelastic range during a major earthquake. The code requirements for building systems, to ensure ductile behavior, depend on the type of materials used (steel, concrete, masonry, etc.) and on the manner by which the system resists seismic inertial forces (moment frame, braced frame, shear wall, etc.). The specific code requirement for each building system will not be presented here. However, the general ductility requirements for some commonly used building systems will be discussed along with any new code provisions related to those systems. The building systems most commonly used for military facilities construction are: masonry shear walls, concrete shear walls, steel braced frames, and moment-resisting frames.

Masonry shear walls

In general, unreinforced masonry has performed poorly when subject to strong ground-motion shaking. The failures of unreinforced masonry has resulted in injury and loss of life to many people during major earthquakes. To prevent the collapse of masonry structures during earthquakes the code requires:

- Sufficient reinforcement, vertically and horizontally, to provide ductility.
- Large factors of safety to minimize the chance of rapid cyclic deterioration and premature brittle failure.
- Diaphragm flexibility limits to prevent brittle failures due to out-of-plane displacements.
- Strong connections to prevent the separation of diaphragms and shear walls.

Concrete shear walls

The code requirements for concrete shear walls follow the same rationale used for masonry shear walls. In addition, the code requires reinforced boundary elements whenever the compressive stress due to the factored code force exceeds 20 percent of the ultimate compressive stress of the concrete. A boundary element is similar to an integral column element placed at the edges and openings of shear walls, with proper confinement reinforcement to prevent concrete spalling during high-intensity load reversals. Concrete spalling leads to a loss of strength and stiffness in shear walls which can ultimately lead to collapse of the wall during a major earthquake.

Steel braced frame systems

In general, it is the intent of seismic codes to ensure that brittle-type failures of systems or components do not occur. Therefore, in braced frame systems the code requires that connections develop the strength of the bracing members in tension or develop a force equal to three times (actually $3R_w/8$ times) the force determined by the equivalent static force method. This ensures that a brittle-type failure of a connection, which could seriously impair the strength of the lateral force-resisting system, does not occur. Braced frames are designed to carry both tension and compression or to carry tension only. The tension-only system however, is a poor energy dissipater. In addition, large displacements usually occur in tension-only systems during major earthquakes and this often leads to damage of building cladding and to damage of other nonstructural and structural components. The SEAOC and UBC as well as the new BDM will limit the slenderness ratio for braced frames in seismic zones 3 and 4. This is to provide tension-only designed members with some capabilities to carry compressive loads without buckling. There are however, exclusions for one- and two-story buildings. These exclusions open the door for the commonly used strap-braced stud wall systems and for rod-braced systems. These particular systems are probably all right for buildings with lightweight cladding. In the new BDM, we intend to provide additional restrictions on the use of strap and rod bracing to ensure this type of bracing can be used only with light weight structures.

The SEAOC, UBC, and new BDM include design provisions for eccentric braced frames. These provisions did not appear in previous codes. The eccentric braced frame combines the desirable drift control features of a concentric braced frame system with the desirable ductile behavior of a moment frame system. A properly designed eccentric braced frame system forces yielding to occur in a ductile link beam rather than by buckling of a brace. Eccentric braced frames also have desirable architectural advantages, meaning the eccentric arrangement of bracing allows more space for door and window openings.

Steel moment frame systems

There have been many significant revisions in SEAOC and the UBC with respect to the seismic design of moment resisting frames. The new BDM will include the new provisions and information on how these provisions are to be applied to the design of special moment resisting frames (SMRF) and ordinary moment resisting frames (OMRF). The intent of the provisions for SMRF's is to ensure:

- That plastic hinges will form in the beams rather than the columns.
- The beam column joint can develop the moment capacity of the beam or the moment capacity corresponding to the development of the panel zone shear strength.

The provisions for OMRF's are less severe. However, an OMRF must be designed for twice the forces of the SMRF's ($R_w = 6$ rather than 12). The design of OMRF's are similar to AISC designs except that the moment connections must be capable of resisting the gravity loads plus 3 $R_w/8$ times the seismic forces.

Quality control and quality assurance

Many of the structural failures that occur during earthquakes can be blamed on poor quality control during construction. Special inspection by a qualified person, independent of the contractor, plays an important role in reducing the incidence of structural failures during major earthquakes. The 1988 SEAOC recognizes this and requires a special design and construction review for essential facilities, hazardous facilities, and special occupancy facilities that are located in seismic zones 2, 3, and 4. Special occupancy facilities are facilities such as schools and day care centers. With respect to the aforementioned facilities SEAOC requires:

- Design review by an independent, licensed structural engineer.
- Specification of an appropriate testing and inspection program by the structural engineer of record.
- Construction observation by the structural engineer of record consisting of:
  - * Review of testing and inspection reports.
  - * Periodic site visits to observe general compliance with the structural engineering plans and specifications. The new BDM will include provisions on QA and QC similar to those of SEAOC. These provisions will be in addition to those now used for the design and construction of military facilities. It is anticipated the new provisions will be difficult to enforce because:
    - ◻ The facility may be at a remote site not easily accessible to the Engineer of Record.
    - ◻ Many designs are shelved and not constructed for many years after the initial design completion.

These new QA-QC provisions, however, are important, and we intend to proceed with the necessary guidance and contractual requirements to ensure that all seismic resisting systems and other architectural, mechanical, and electrical systems important to life safety are constructed with the seismic protection features intended by the Engineer of Record.

References

Applied Technology Council Publication ATC 3-06. 1978. "Tentative Provisions for the Development of Seismic Regulations for Buildings."

Freeman, Sigmund A. 1979. "Seismic Design Criteria for Multistory Precast Prestressed Buildings."

International Conference of Building Officials. 1988.

Seismology Committee of the Structural Engineers of California. 1988. "Recommended Lateral Force Requirements."

Wiss, Janney, Eistner Associates, Inc. 1990. "Intermediate Draft for the Total Revision and Update of TM 5-809-10, Seismic Design for Buildings."

Appendix D
Conference Attendees

LIST OF ATTENDEES - BY NAME

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Appendix E
Structural Steel Connection Design on Federal Projects

Structural Steel Connection Design on Federal Projects

by
David B. Ratterman[1] *and Stephen E. Smith*[2]

Remarks by Mr. Ratterman

There is a broad difference of opinion in the United States on the subject of delegated or shared responsibility for connection design on buildings with structural steel frames. This difference of opinion exists in both the engineering community and in the construction industry. As is true in many facets of construction practice, economic considerations drive this difference of opinion.

Mr. Smith and I are attorneys in private practice in Louisville, Kentucky. We represent owners, developers, general contractors, specialty contractors, bonding companies, and other interests in the construction industry. The construction industry comprises about 90 percent of our practice. The American Institute of Steel Construction (ASCI) is one of our clients.

We are going to discuss the legal and practical problems related to the connection design issue in the context of current Corps of Engineers contract provisions. We will discuss the AISC policy on this issue, what we understand to be the proposed Corps of Engineers policy on this issue, and why we feel those policies to be in the best interest of design professionals, contractors, project owners, and the public in general.

The American Institute of Steel Construction was founded in the 1920's as a nonpartisan organization of engineering and construction professionals dedicated to safety, uniformity, and economy in the application of steel construction in the United States. In the ensuing 60+ years, AISC has sponsored broad-ranging research and educational programs related to steel construction and has published *the definitive* construction manuals, specifications, and codes on these subjects. I am sure all of you are very familiar with the *AISC Manual of Steel Construction*, which is currently in its 9th edition, and its more recently published companion work, the *LRFD Manual*.

From the first edition of the *Manual of Steel Construction* through the 9th edition, AISC's position on responsibility for connection design has been very clear and very consistent. Quality and safety of the constructed project must be ensured. The integrity of the constructed project can only be maintained by the engineer who has designed the primary structural system. For the purposes of this presentation we will call that person the "engineer of record."

The engineer of record can be aided in connection design on complex projects by input from a steel fabricator, but the fabricator will never be aware of all factors considered by the engineer of record in the overall design concept. For a construction project to run smoothly, and for safety to be ensured, the limits of the respective authority and responsibility of all parties involved in steel construction, together with the applicable submittal review procedure, must be established in very clear contract language.

In 1985 the Boards of Directors of the American Institute of Steel Construction and the American Society of Civil Engineers issued a joint "white paper" entitled "Final Report and Recommendation on Assignment of Authority and Responsibility for Design of Steel Structures." That document remains the

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[1] General Counsel, American Institute of Steel Construction.
[2] Attorney, Goldberg & Simpson, P.S.C., formerly District Counsel, U.S. Army Engineer District, Louisville.

official policy statement for both organizations. In pertinent part, it reads as follows:

> "The contractual arrangement for design that offers the best control of structural integrity is one under which the EOR has responsibility and authority for the entire structural design, including connections."

We understand that the proposed Corps of Engineers guide specification will provide that complex connections shall be fully designed on the contract drawings and that any connections not fully designed on the contract drawings must be capable of development directly from the AISC specification.

In adopting this approach, the Corps of Engineers is in good company. Not only does this approach comport with the policy enunciated above, but it also comports with the policy of the Coalition of American Structural Engineers, the statutory guidelines of the states of New York and Florida, the Structural Engineers Associations of Illinois and California, the Building Code of Northern Virginia, and many other model engineering licensing codes and state regulations.

I would like to talk briefly about why this position is correct from a practical standpoint and why its implementation is important to contractors, designers, owners, and the public in general.

Robert Frost wrote that "good fences make good neighbors." This principle is never more relevant than in the area of construction contracts. It is essential that all elements of the agreement among the parties involved in a construction contract be unambiguously set out in writing.

Once a problem develops on a construction project it is too late to attempt to understand the meaning of the contract provisions. All too often one finds after a problem has developed that contract terms thought to provide "iron clad" protection are ambiguous and leave parties exposed to painful consequences.

Nowhere is the potential harm which can be caused by ambiguities in contract language more severe than in the area of structural connection design. We believe that many current attempts to delegate or share the responsibility for connection design, including contract language found in some Corps of Engineers contracts, create substantial ambiguity because they leave unclear exactly which party bears responsibility for this critical element of the primary structural system.

The arguments against formal shared responsibility for connection design are many. Time does not permit us to delve into each of these arguments in detail but, briefly, I would like to list some of the arguments against the shared responsibility concept:

- It places "two cooks in the kitchen."
- It runs contrary to existing law and established practice.
- There is a dearth of clear contract terms existing in the industry specifically defining the limits of responsibility of various parties involved—it may be impossible to adequately define these limits if responsibility is truly to be shared.
- It creates potential conflicts of interest on the construction project and in the engineering community.
- It is a potential violation of public works statutes prohibiting selection of design professionals by competitive bidding. In particular, on Federal projects, I believe it violates the so-called Brooks Act.
- It is a potential inhibition to open competition, ultimately increasing the cost of construction and the price of the finished product to the consumer.

Everyone in this room is undoubtedly familiar with the Kansas City Hyatt Regency disaster in which a skywalk collapsed and in excess of 100 individuals lost their lives. The Hyatt disaster was caused by a faulty connection which was not discovered in advance at

least in part because of a question in the minds of the project team as to who had responsibility for connection design.

The administrative hearing judge for the Missouri Professional Engineers' Licensing Board and the Court of Appeals of the State of Missouri studied the causes of this disaster, the contract documents, and the underlying law of Missouri and other jurisdiction in great detail. The opinion which discusses the legal and technical issues involved in this disaster exceeds 400 pages in length. In that opinion, commenting on Missouri law, the administrative hearing judge concludes that:

> "While the engineer of record may properly delegate the work of performing engineering design functions, he cannot delegate his responsibility for the structural engineering design where it concerns professional engineering functions. This responsibility is non-delegable."

Let's talk about contractual responsibility on construction projects and the practical problems involved in trying to delegate design functions to construction contractors.

Figure 1 is a "wiring diagram" of a typical construction project. The solid lines indicate formal contractual relationships. This is the type of wiring diagram that would have existed on the Hyatt Regency project and that exists on most Corps of Engineers projects.

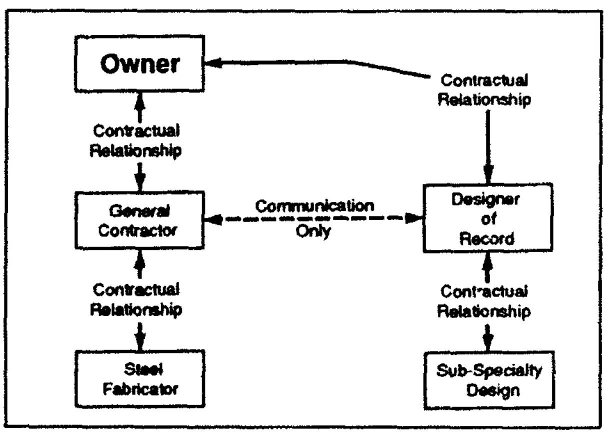

Figure 1.

You will note that this diagram consists of two distinctly separate "branches." The right-hand branch involves the design team, the left-hand branch involves the construction team.

The Hyatt Regency opinion discusses these two separate teams in some detail. It very clearly delineates the historical approach to construction contracting in the United States and the rationale behind that approach. Under that approach the design process is kept separate by contract from the construction process. It can be argued that economic forces may cause the construction team to be concerned with economy and speed of construction while the engineering team should be concerned solely with the quality of construction and soundness of the end project. Therefore, to remove economic pressure from the realm of quality and soundness of construction, each branch reports separately, directly, and independently to the construction owner. To intermingle the functions of these two teams may court disaster. In the logic of the Hyatt Regency decision and many other legal commentators, such intermingling may in and of itself violate professional licensing statutes.

Let me now bring to your attention some typical contract language dealing with fabricator design responsibility found in current Corps of Engineers construction contracts.

At least one Corps District has issued a contract containing the following language:

> "...connections ... shall be designed by the fabricator and the calculations shall be submitted with the shop drawing."

Example 2 comes from another District:

> "...all connections shall be designed by the contractor...submit...all calculations...signed [and sealed] by a professional engineer licensed in the state of ________. Review of shop drawing... will not in any way relieve the contractor from the responsibility for the adequacy of the design of the connections..."

Typical Corps of Engineers contracts also contain fairly standard provisions substantially limiting the scope of shop drawing submittal review to "general conformance with contract terms only" and "not intended to be a complete check."

I submit to you that these clauses, read in conjunction with the preceding clauses, tell the contractor that *it* has full authority over connection design. Authority and responsibility, as you know, go hand in hand and cannot be separated. I believe the clauses cited above take that authority and responsibility away from the engineer of record and give it to the steel fabricator. I don't think that is what you want, but I believe that is what you've got because of language which may have inadvertently found its way into some of your structural steel specifications.

The legal elements necessary for professional design responsibility include a license in the state of the project and complete control over design or complete review of design calculations performed by others. Under the clauses cited above, the engineer of record does not meet either of these two elements; but the fabricator does. I would ask, rhetorically, whether the fabricator's professional engineer can bear the design responsibility for connections without exercising an equal degree of design authority? I think not. I would also ask whether the engineer of record can exercise design authority without bearing an equal degree of responsibility? Again, I think not.

The technical elements necessary to perform connection design include knowledge of the applicable building codes, knowledge of the overall design concept of the project, loading data, and knowledge of current industry practice, technology, and state-of-the-art design theory.

I would say to you, and I believe most of you would agree, that even if you filled this large meeting room with contract documents, you could never include all the elements of practice, technology, theory, and assumptions underlying the design of a complex structure. Simply put, reasonable engineering minds can differ as to the proper approach to a complex design problem.

Let me ask you two more rhetorical questions. Who bears the risk of an ambiguous contract term? You do, the party who drafted the contract. Who bears the economic risk of a professional difference of opinion related to individual engineering office practice? You bear that risk if you are the party without design responsibility.

In the case of the current Corps of Engineers contract provisions which I quoted above, this means that you, the owner, the Corps of Engineers, the party who drafted the contract, bear the responsibility and the *expense* of a professional difference of opinion between your structural steel fabricator and the A/E who prepared the overall structural design.

Under this current contract language, the fabricator has a right to rely on its ability to proceed with fabrication of any reasonable connection design which complies with the contract documents upon which it based its competitive bid, regardless of whether the engineer of record would have designed the connection in the same manner. If the engineer of record or the owner wants the design changed (and please don't misunderstand me, you have the right to require that the design be changed), then I believe the fabricator is fully within its rights to expect to be paid to change its design. All of this is above and beyond the very crucial threshold questions of safety. Shouldn't the party who designed the entire structure be responsible for all of its elements? Isn't that the cleanest, safest, simplest approach?

The heart of the problem is presented in Figure 2, another project wiring diagram. As this diagram indicates, traditional contract documents simply do not deal effectively with the "unnatural" relationship between a steel fabricator and an engineer of record when the fabricator is tasked with responsibility of

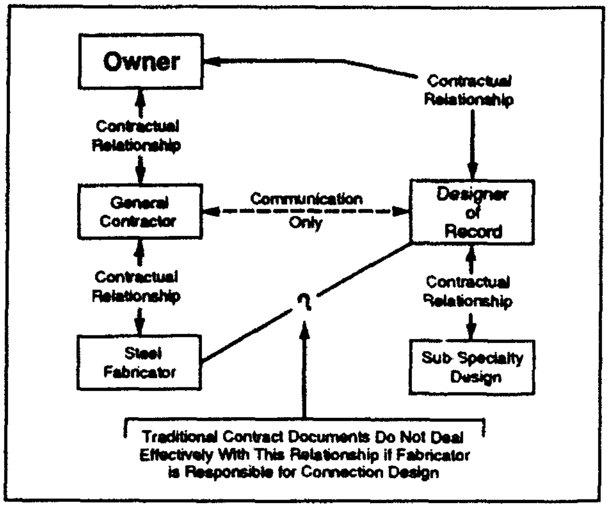

Figure 2.

designing a critical element of the primary structural system.

This problem is doubly significant on Federal construction projects because, in my estimation, it violates the provisions of the Brooks Act, 40 USC 541-44. The Brooks Act provides in pertinent part that design professionals are to be prequalified on Federal projects; they are to be screened by an evaluation board for consideration on projects which have been specifically advertised. Design professionals are ultimately to be selected on the basis of discussions and competitive negotiations based on qualifications alone; by Federal regulation price considerations are to be specifically excluded from the evaluation board process.

When you have a steel fabricator professionally responsible for the design of connections on a complex construction project, how has that professional design responsibility been awarded? On the basis of competitive negotiations and qualifications? No. While you may have the most highly qualified fabricator in the United States designing your connections, that design service has not been awarded in accordance with the Brooks Act but, rather, has been awarded on the basis of the lowest competitive bid.

The Brooks Act, of course, is a matter which is peculiar to Federal construction contracts. The overall problem is the systematic problem addressed in Figure 2, and this problem pervades all construction projects.

The only logical approach to preventing the potentially disastrous effect of contract ambiguities on this issue is to adopt what has been the traditional approach of the industry for at least the past 65 years, the approach sanctioned by the overwhelming majority of professional societies and licensing boards in the United States and the approach proposed by the Corps of Engineers—to vest full authority and responsibility for connection design in a single entity which is not a direct participant in the construction chain-of-command—the professional engineer of record who serves as an independent consultant to the construction owner.

Since I know that many of your questions will concern matters which are peculiar to Corps of Engineers policy and practice, I will now turn the presentation over to my colleague, Stephen E. Smith, who recently left Government service after 15 years of dealing with Corps construction contracts. In the last 6 years of his service, Mr. Smith served as District Counsel for the Corps' Louisville District.

Remarks by Mr. Smith

Very soon after leaving the Government, the problem with the shifting of design responsibility from the engineer of record to the fabricator became a major issue with our law firm. When the problem found its way to a case involving a Corps of Engineers contract, my initial advice to Mr. Ratterman and the client was that the Corps did things differently than the private sector and that the action taken by the Corps may have been a matter of national policy. After gaining a better understanding of the issue and the facts of the particular case we were involved in, I was convinced that the personnel of the Corps simply did not fully understand the implications arising from the shifting of design responsibility nor was the action taken consistent throughout the county.

Over and above any client-driven needs to address this problem, my personal concern was that the Corps was allowing actions to be taken within their contractual relationships with their architects that were clearly not in the best interests of the Government. As we had seen in several specific cases, it is often very difficult to discover that problems which manifested themselves in delays in field construction can be traced to shop drawing review problems back at the architect's office.

This problem is exacerbated by the basic Corps of Engineers structure with regard to lines of responsibility between Construction and Engineering Divisions. The fact that the design contract was the province of the Engineering Division and the work is being performed under the supervision of the Construction Division ensures that there is less than perfect communication between the divisions. If design responsibility problems manifest themselves as field erection problems, the solution to the problem will be more difficult to achieve without close communication between the two elements.

It is still an open question whether the new alignment into a Project Management Division will serve to end some of the problems which arise due to the lack of communication between Construction and Engineering Divisions during construction. A longer track record is necessary before that question can be answered.

This is not to say that there is not a cooperative, informational exchange between the divisions during the construction process; only that with the separation of responsibility between the elements, subtle problems are not as easily recognizable. The difficulty arising from different funding sources for the design phase versus the construction phase has led to problems in many construction projects. Close communication among all elements of the Corps' team must take place at the design stage to reduce problems which arise in situations alluded to previously. This communication must continue through the life of the project between those responsible for design and those responsible for construction.

The message I am conveying to the Corps is rather simple: do not allow problems under your design and construction contracts to arise by accident. Make sure that you, individually, understand the contract language and that the contract is clear and unambiguous. Go to your lawyers and ask for clarification if something written by an architect in the specifications is not clear.

Mr. Ratterman has conveyed to you the position of AISC on design responsibility, and I understand that yesterday you were provided guidance by the Office of the Chief of Engineers on this matter.

Make no mistake about it, the simplest and best position for the Corps to take is that design responsibility may not be shifted to the fabricator by the design engineer. But if by some mechanism, design responsibility has been shifted to the construction contractor, intelligent administration of the construction contract will help to reduce the Government's exposure.

Mr. Ratterman addressed the fact that shifting design responsibility to the construction contractor may be a violation of the Brooks Act. When this aspect of the problem was first raised in our office and later at the Chief of Engineers' Office, I had some reservations about that position. However, as I have evaluated the issue in more detail, I find that there may be validity to the assertions by Mr. Ratterman.

I know that some Corps representatives take the position that there is a certain amount of design responsibility in every construction contract—metal building contracts, for example. The question is at what point do you cross the line from an allowable degree of contractor design to a violation of the Brooks Act. The Brooks Act was ostensibly passed in an effort to remove economic considerations from design criteria which would in turn lead to the design of safer buildings. Placing design responsibility on the fabricator who is selected on strictly economic terms is the very scenario which the Brooks Act was meant to

prevent. I have come to the point in my thinking that I believe that placing design responsibility on the fabricator is a violation of the Brooks Act.

In conclusion, I believe that the Corps is wise in addressing the issue of design responsibility on a national basis. I believe that the position enumerated to you yesterday is sound, and will result in more economical projects in the future. We would like now to answer any questions which you may have on anything which Mr. Ratterman and I have discussed this morning.